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PRESTRESSED CONCRETE FENDER PILES: FINAL DESIGNS

ABSTRACT Drawing on prior Naval Civil Engineering Laboratory test results, final pile design criteria are established. Design aids are developed for a range of pile cross-section sizes and configurations, concrete strengths, and span lengths. Detail drawings and specifications are presented for the final pile designs.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043

METRIC CONVERSION FACTORS

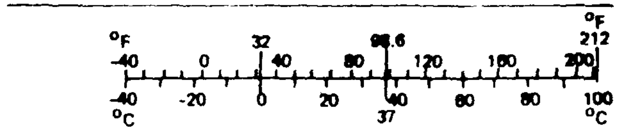
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Drawing on prior Naval Civil Engineering Laboratory test results, final pile design criteria are established. Design aids are developed for a range of pile cross-section sizes and configurations, concrete strengths, and span lengths. Detail drawings and specifications are presented for the final pile designs.

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PRESTRESSED CONCRETE
FENDER PILES: FINAL DESIGNS

Prepared for

Naval Civil Engineering Laboratory
Port Hueneme, California

ABAM Engineers Inc.
33301 Ninth Avenue South
Federal Way, Washington 98003-6395

with

Naval Civil Engineering Laboratory
Port Hueneme, California 93043

and

Ben C. Gerwick, Inc.
500 Sansome Street
San Francisco, California 94111

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SECTION 1
SUMMARY

The Department of the Navy, through the Naval Civil Engineering Laboratory (NCEL) at Port Hueneme, California, has initiated a program to develop prestressed concrete fender piles for use at Navy piers in a wide range of fendering applications. This report completes the analysis portion of the multiphase testing, analysis, and design effort [1.1].

NCEL's final test pile program verified the revised analytical model developed in earlier phases and provided valuable cyclic performance data for this analysis phase [1.2]. The analytical model successfully predicted the test pile performance for monotonic loading up to the assumed ultimate strain of the concrete. Post-elastic behavior of the piles was ductile up to pile failure at greater than two times the allowable design energy.

Major tasks in this phase included establishment of pile design criteria, development of design aids for selected pile configurations, preparation of detail drawings and specifications for selected pile configurations, and preparation of details for rubstrip attachment to the piles. Other tasks in this phase were to verify the final test pile results and prepare cost estimates for selected pile configurations.

Pile performance criteria were established based on the previous test pile programs [1.3]. The pile design procedure was further refined in this phase to limit residual crack widths, thereby mitigating possible corrosion damage. The baseline pile recommended as a result of this latest phase is very similar to the test pile Mark No. 5 (MK 5) except that the reinforcement and its placement have been optimized to achieve a working energy level of approximately 20 ft-kips with a residual crack width less than 0.012 in. after 100 cycles of working level load applications.

Design aids were developed for a range of pile sizes and configurations, concrete strengths, and span lengths. These design aids are in the form of easy-to-use graphs with supplemental tables for use with nonstandard designs. Connection details from an earlier report for the ultra high molecular weight (UHMW) plastic rubstrip were refined.

Silica fume was deleted from the baseline pile because of its significant cost impact. Silica fume may be required in certain geographical locations to increase durability of the concrete and to provide high-strength concrete; this decision is best made by the facility designer.

As a result of this phase, design aids and detailed drawings and specifications were prepared to assist the Department of the Navy in procuring precast prestressed concrete fender piles for Navy facilities.

SECTION 2
INTRODUCTION

2.1 SCOPE OF WORK

The major effort in this phase was directed toward establishing design aids for the prestressed concrete fender piles. The final NCEL test pile results were reviewed and compared to the latest version of the ABAM computer program, FENDER, and compared very well [1.3].

Pile performance criteria should be based on structural and durability parameters. The structural parameters were defined in previous phases but required some modification. The durability parameters, specifically residual crack width and compression zone fatigue, could only be determined from cyclic testing. Review of the test pile data resulted in recommendations to modify the structural parameters: concrete strain, and steel stress. The new recommended design values are $\epsilon_c \leq 0.0021$ in./in. and $f_{pw} \leq 210$ ksi for working level energy. These values were used in the development of the design aids. Another important finding in the cyclic testing program was that confining reinforcement using deformed rebars can significantly reduce the residual crack width on the tension face of the pile when compared with smooth wire. Hence, the possibility of reinforcement corrosion damage can be mitigated by the use of deformed rebar.

2.2 BASELINE PILES

Several baseline piles were used in this phase of the work. The original baseline pile was recommended in the previous analysis report [1.3] and consisted of

- o 18-in.-sq piles
- o 65-ft span
- o Sixteen 1/2-in.-diameter 270-ksi prestressing strands
- o f'_c = 8000 psi
- o f_{pc} = 450 psi
- o Square confinement reinforcement

Piles of this type were tested in the first part of the final test pile program and showed poor cyclic performance. The actual test piles had a design span of 30 ft and used double wraps of smooth wire for confinement reinforcement. These piles had larger than desired residual crack widths after cyclic load testing.

Subsequent analysis resulted in recommendations to fabricate five additional test piles to the following specifications:

- o 18-in.-sq piles
- o Span suitable for testing
- o Twenty 1/2-in.-diameter 270-ksi prestressing strands
- o f'_c = 8000 psi

- f_{pc} = 600 psi
- Square and circular confinement reinforcement

This specification is very similar to test pile MK 5 from the preliminary test pile program [2.1] except that the reinforcement has been optimized. Two of the test piles from this second group exhibited very good cyclic performance. As a result, the earlier baseline pile was replaced with two new baseline piles, one with a square and one with a circular reinforcement configuration. One significant recommendation is the use of No. 3 deformed rebar for confinement steel in the square tied piles and either smooth W11 or No. 3 deformed rebar for the circular tied configuration. Development of the design aids and cost estimates was based on the latter two baseline piles.

SECTION 3
PILE TESTING

3.1 TEST PILE PROGRAM

NCEL conducted the latest testing in two different phases. In the first phase, 16 piles were fabricated and tested. Typically, the piles consisted of 16 strands with an effective prestress (f_{pc}) of 450 psi. The variables were the type and spacing of confining reinforcement. The piles are referred to as MK 9 through MK 24 and are discussed in further detail in a NCEL report [1.2]. In addition, an undamaged section of pile MK 5 from the preliminary test pile program was also tested.

Based on the test results of the first phase, an additional five piles were fabricated and tested (see Figure 3.1). These piles had 20 strands with an effective prestress (f_{pc}) force of 600 psi. One of the five piles had a circular strand pattern while the other four had a rectangular pattern on two faces. The piles in this second phase of testing are referred to as MK 25 to MK 29. The results for the second phase of testing are reported in Table 3.1 and Figure 3.1 [3.1].

3.2 COMPUTER MODEL VERIFICATION

A computer program, titled FENDER, was used to analyze piles of different cross sections and strand configurations. FENDER was developed and written in the previous phase of this study [1.3].

The analytical model used as the basis for the program takes into account the inelastic properties of concrete, prestressing strand, and reinforcing steel. It is based on the assumptions that plane sections remain plane and that deformations are small. For a given cross section, the moment-curvature relationship is found using strain compatibility between the concrete and steel. The ultimate moment capacity of the cross section is based on a maximum concrete strain of 0.003 in./in. The energy per unit length for the cross section is based on the area under the moment-curvature curve for the moment applied over the unit length. The summation of the unit energy along the length of the pile gives the total energy that the pile can absorb. The calculation of the pile deflection is based on the energy-to-load relationship. See Ref. 1.3 for further details.

The basic assumptions used in the program that the designer should be aware of are summarized below:

- Maximum allowable concrete strain at nominal strength is 0.003 in./in.
- Concrete is assumed to have no tension capacity.

- Concrete stress block is trapezoidal in shape at high concrete strains.
- Modulus of elasticity of concrete is based on work from the previous study [1.3].
- Yield stress is $f_{py} = 0.90 \times f_{pu}$ for low relaxation strand and $f_{py} = 0.85 \times f_{pu}$ for stress relieved strand [3.2].
- Modulus of elasticity is $E_s = 28,000$ ksi.

In order to recheck the accuracy of the computer program FENDER, a comparison was made between the computer results and the test data provided by NCEL (Reference 1.2 and supplemental test pile results in Figure 3.1 and Table 3.1). The program had already been verified in the previous analysis report [1.3]. The most straightforward test data to compare was the load verses deflection of the pile at the point of load application. The five piles chosen for the comparison are listed and discussed below.

a) MK 12: 18-in.-sq pile with 16 strands in a rectangular pattern. Effective prestress of 450 psi and confinement steel of W5 square spiral at 3-in. pitch. This pile was cyclically loaded. See Figure 3.2 for comparison of results.

b) MK 19: 18-in.-sq pile with 16 strands in a rectangular pattern. Effective prestress of 450 psi and confinement steel of W5 square spiral at 6-in. pitch. This pile was cyclically loaded. See Figure 3.3 for comparison of results.

c) MK 22: 18-in.-sq pile with 16 strands in a rectangular pattern. Effective prestress of 450 psi and confinement steel of two W5 square spirals at 3-in. pitch and additional cross ties. This pile was monotonically loaded. See Figure 3.4 for comparison of results.

d) MK 28: 18-in.-sq pile with 20 strands in a rectangular pattern. Effective prestress of 600 psi and confinement steel of No. 3 rebar ties at 3-in. spacing. This pile was cyclically loaded. See Figure 3.5 for comparison of results.

e) MK 29: 18-in.-sq pile with 20 strands in a circular pattern. Effective prestress of 600 psi and confinement steel of two W5 circular spirals at 3-in. pitch. This pile was cyclically loaded. See Figure 3.6 for comparison of results.

The existing computer program provides good overall results. The computer output is comparable to the monotonic test results, but the cyclic behavior cannot be predicted. When the pile was cyclically loaded to approximately 80% of its expected nominal load, the cracking along the pile resulted in a small offset of the specimen after two load cycles. Hence, no direct comparison can be made between the final positions of the two curves. What is important to note is that the two curves are almost identical to one another in the initial stages. As was shown in earlier phases of the fender pile study, the equation of the curve (in other words, the shape of the curve) did not vary with the concrete strength. Only the ending point of the curve changes with concrete strength. Therefore, the computer model appears to accurately predict the shaped of the load-deflection curve of the pile. But the end point of the curve, representing the ultimate strength of the pile, can only be predicted for a one-time, monotonic load.

3.3 SUPPLEMENTARY TEST PILE RESULTS

The failure mode for the supplementary group of test piles was different from previous results. The strands did not rupture all at once. Because of the lower concrete strengths, the pile cross-section configuration did not have a balanced condition at failure; if the concrete strength were greater, there would have been less damage to the compression face at the low load levels.

The serviceability of the pile is a very important consideration in the design of the prestressed concrete fender pile. The deflection of the pile is not as important as the crack widths on the tension face or the tendency for the cover to spall on the tension side of the pile. Crack width is apparently a function of load level. The maximum allowable residual crack width under working loads was established at 0.012 in., based on the test results and previous work [1.3]. Test observations revealed that loads above a cyclic strain of 0.0021 in./in. resulted in cumulative crack growth and compression zone damage. Cyclic strain was defined as (measured strain) - (residual strain).

The testing of the supplementary piles, MK 25 to MK 29, provided information on the effect of confinement steel on the crack width in the pile. The type of reinforcing used makes a significant difference in the residual crack width that occurs due to the cyclic loading. The two different types of reinforcing used for the rectangular strand pattern were two W5 spiral wires and one No. 3 rebar tie. The piles that contained the smooth wire in a square spiral experienced larger residual cracks than did the pile with the No. 3 deformed rebar. The reason for the deformed rebar performing better than the smooth wire is thought to be due to the lugs on the rebar itself. They grip or anchor the concrete better than the smooth wire.

The two piles that performed the best in the supplementary testing were MK 28 with a rectangular reinforcing configuration and MK 29 with a circular reinforcing configuration. For a given strain, both piles performed well but the circular reinforcing configuration pile had a lower load but larger residual crack widths because of a larger deflection. Despite this, it was decided to include the circular configuration in the final recommendations because of possible cost savings in pile fabrication.

3.3.1 Crack Width

The issue of concrete cover, relating to the crack width in the pile, has two opposing views. The further away the steel is from the surface the larger the cover will be to protect it. But the corresponding crack width will be larger because the steel is less effective in resisting the force. On the other hand, the closer the steel is to the surface, the less distance the water will have to travel to reach the steel. But the steel is more effective so the crack width is smaller.

The rectangular strand pattern has more area of steel close to the surface than the circular strand pattern. In the circular strand pattern, only two strands are close to the surface, while the other strands are located at further distances from the concrete face. The rectangular strand pattern has all of the first layer of strands close to the surface. Table 3.2 presents a summary of the load, cycle, and resulting maximum residual crack width for test piles MK 28 and MK 29.

The test data show that for approximately the same load and number of cycles, the pile with a rectangular strand pattern (MK 28) has a smaller residual crack width than the pile with a circular strand pattern (MK 29). After 150 cycles of a 39-kip load, the pile with a rectangular pattern has a residual crack width of 0.009 in. compared to the pile with a circular pattern having slightly less load (38 kips) for 150 cycles but a crack width of 0.015 in. It must be noted though, that the circular pattern is less efficient in resisting a given load; hence, the concrete compressive strain is slightly larger than with the rectangular pattern.

3.3.2 Compression Zone Behavior

The compression strain limit of 0.0021 in./in. in the piles was chosen to prevent compression zone fatigue damage due to cyclic loads. It was observed during the testing that deterioration of the pile occurred when the compressive strain was higher than this value. The damaged compression zone decreased the energy that the pile could absorb. Monotonically loaded piles could withstand compressive strains in excess of 0.003 in./in. prior to compression zone spalling.

However, the nominal capacity of the pile is theoretically reached when the concrete compression strain reaches 0.003 in./in. At this point, it is expected that the concrete cover would spall. The pile would not be able to carry the same magnitude of load but would continue to carry some load as it deflects. As the pile deflection increases, the strands would continue to yield until they finally rupture, unless the concrete strength is too low and the pile forms a hinge without the strands ever rupturing.

3.3.3 Reinforcing Configuration Comparison

The mode of failure is similar between the circular and rectangular reinforcing configuration piles up to the point at which the strands finally rupture. For higher strength concrete, there is spalling of the cover and a hinge develops in the region of the applied load. When loaded further beyond the nominal capacity, the strands in the rectangular pattern rupture all at once, whereas the circular pattern experiences an "unzipping" effect. The strands nearest the surface rupture first, followed by the next closest strands to the surface as the pile is deflected further.

3.4 PILE PERFORMANCE

The performance of a fender pile under a monotonic loading can be illustrated with Figure 3.7. The example pile selected was MK 16, an 18-in.-sq pile with 16 strands. The closely spaced cross-hatched area under the load deflection curve represents the working energy capacity of the pile. It is limited by a maximum concrete compressive strain of 0.0021 in./in. At this point, there should be no damage to the compressive face and the crack widths under cyclic load would be in the acceptable range.

As the pile is loaded further, the top of the curve represents the point where the concrete cover spalls. The area under the curve to this point is the nominal energy capacity. The pile may have to be replaced after loading to this level.

Beyond this point, the pile cannot maintain the same magnitude of load but will continue to deflect and absorb energy until the strands rupture. The area under the entire curve represents the failure energy of the pile, E_f. The total energy at failure is more than two times the nominal energy.

The performance of a fender pile under cyclic loading can be illustrated with Figure 3.8. The example pile selected was MK 12, an 18-in.-sq pile with 16 strands. This pile was loaded in three distinct intervals: Load 1 (concrete strain of 0.00155 in./in.) for 300 cycles, Load 2 (concrete strain of 0.00185 in./in.) for 200 cycles, and Load 3 (concrete strain > 0.003 in./in.) to failure. There is some "softening" of the concrete as the cycles at each load level increase, with a small decrease in energy capacity of the pile. It should be noted that cyclic loading does not appear to affect the nominal load carrying capacity of the pile (see Figure 3.7) when concrete compressive strains are less than 0.0021 in./in. However, there will be a slight loss in nominal energy capacity.

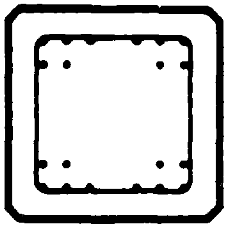
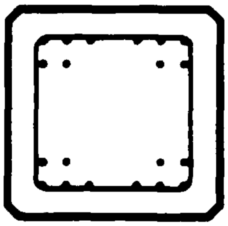
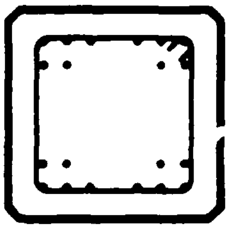
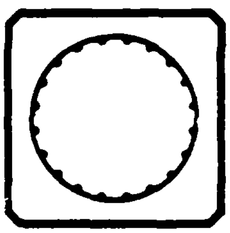
TABLE 3.1
SUPPLEMENTARY NCEL TEST PILE RESULTS, PILES NK 25 THROUGH NK 29

Pile NK	Concrete Strength f'_c (psi)	Load P (kips)	Cycle	Maximum Residual Crack Width (0.001")	Measured Concrete Strain $\varepsilon_c \times 10^{-6}$ Max	(Residual)	Model Concrete Strain $\varepsilon_c \times 10^{-6}$	Measured Energy per Cycle (ft-kips)	Model Energy per Cycle (ft-kips)	Model Steel Stress (ksi)	Compression Zone Damage
25	7360	43.4	1	4	1832	(0)	2200	10.5	8.9	202	No
			50	14	2130	(186)		9.4			
			100	20	2292	(260)					
			150	23	2367	(335)					
			200	29	2404	(372)					
		51.5	last	-	-	-	-	87.8	-	-	-
26	7300	38.0	1	8	1593	(0)	1786	6.5	6.2	174	No
			50	9	2016	(126)		6.1			
		40.0	50	12	2452	(322)	1933	6.7	7.1	184	No
		42.0	50	21	2827	(536)	2100	7.8	8.1	195	No
		44.0	50	27	3207	(431)	2282	8.8	9.3	206	No
		46.0	11	-	-	-	2471	9.6	10.5	217	Yes
			last	-	-	-	-	81.5	-	-	-
27	7190	40.0	1	6	1711	(66)	1958	7.8	7.1	185	No
			50	11	2160	(261)		7.1			
			100	15	2393	(374)					
			150	20	2505	(457)					
		42.0	50	26	2766	(616)	2127	8.4	8.2	196	No
		44.0	50	31	2994	(682)	2290	9.1	9.4	206	Yes
			100	36	-	-					
			117	-	-	-					
			last	-	-	-	-	72.0	-	-	-
28	7070	39.0	1	2	1578	(0)	1893	6.8	6.7	180	No
			50	4	1747	(64)		6.3			
			100	7	1950	(68)					
			150	9	2044	(150)					
		42.0	50	11	2307	(188)	2155	8.2	8.2	196	No
			100	13	2382	(188)					
		44.0	50	17	-	-	2340	9.1	9.4	207	Yes
			100	18	-	-					
		46.0	50	24	-	-	2550	10.3	10.7	218	
			100	34	-	-					
			148	-	-	-					
			last	-	-	-	-	72.2	-	-	-
29	7420	38.0	1	4	1644	(37)	2081	8.1	7.2	195	No
			50	8	1938	(98)		7.2			
			100	10	2049	(112)					
			150	15	2124	(167)					
		40.0	50	23	2165	(130)	2300	8.8	8.4	209	Yes
			100	27	-	-					
		42.0	50	33	-	-	2543	9.4	9.7	221	
			52	-	-	-					
			last	-	-	-	-	56.8	-	-	-

Notes: 1) Supplementary test pile results as reported by NCEL [3.1]
2) Piles fabricated on 19 June 1987
3) Piles tested in late July 1987

TABLE 3.2
COMPARISON OF CRACK WIDTHS
BETWEEN PILES MK 28 AND MK 29

Pile	f'_c (psi)	Load (kips)	Cycle No.	Residual Concrete Strain	Maximum Residual Crack Width (in.)	Measured Energy (ft-kips)
MK 28	7070	39	1	0.00158	0.002	6.8
			50	0.00168	0.004	6.3
			100	0.00188	0.007	
			150	0.00189	0.009	
		42	50	0.00212	0.011	8.2
			100	0.00219	0.013	
MK 29	7420	38	1	0.00161	0.004	8.1
			50	0.00184	0.008	7.2
			100	0.00194	0.010	
			150	0.00196	0.015	
		40	50	0.00204	0.023	8.8
			100	---	0.027	

	PILE MARK	CONFINEMENT REINFORCEMENT
	MK-25 MK-26	(2) - W5 SPIRAL @ 3" PITCH
	MK-27	(1) - W5 SPIRAL @ 3" PITCH
	MK-28	#3 TIES @ 3" SPACING
	MK-29	W11 SPIRAL @ 3" PITCH

DESIGN DATA AND PROPERTIES COMMON TO ALL FIVE PILES.

- PILE SIZE: 18" SQUARE
- CONCRETE STRENGTH: f'_c = 8 ksi
- STRANDS: 20-1/2" DIAMETER 270 ksi STRANDS
- CONCRETE COMPRESSIVE STRESS: f_{pc}=600 psi

FIGURE 3.1, SUPPLEMENTAL TEST PILES AS REPORTED BY NCEL [3.1]

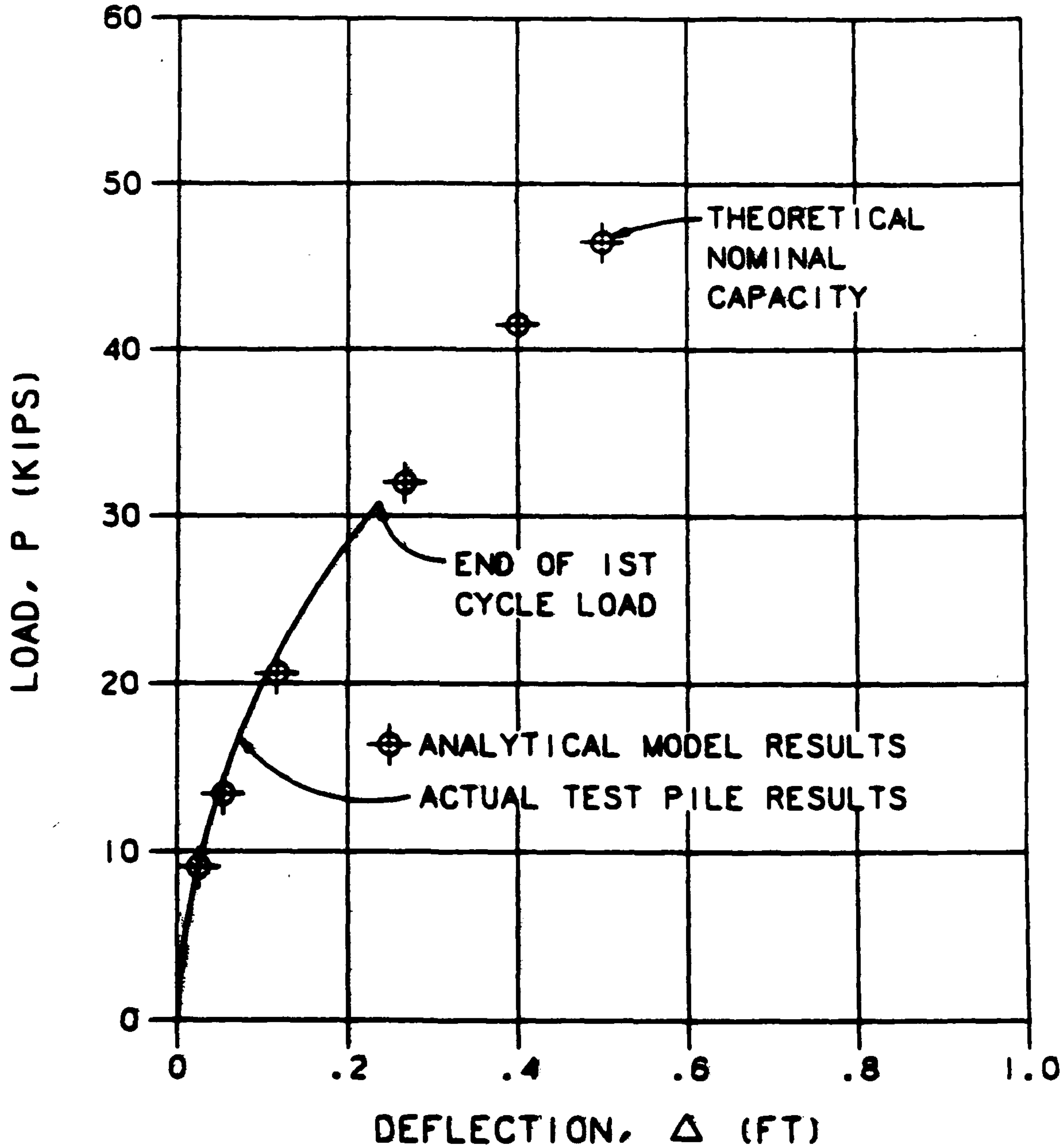


FIGURE 3.2, COMPARISON OF ANALYTICAL MODEL WITH NCEL TEST PILE MK-12 RESULTS

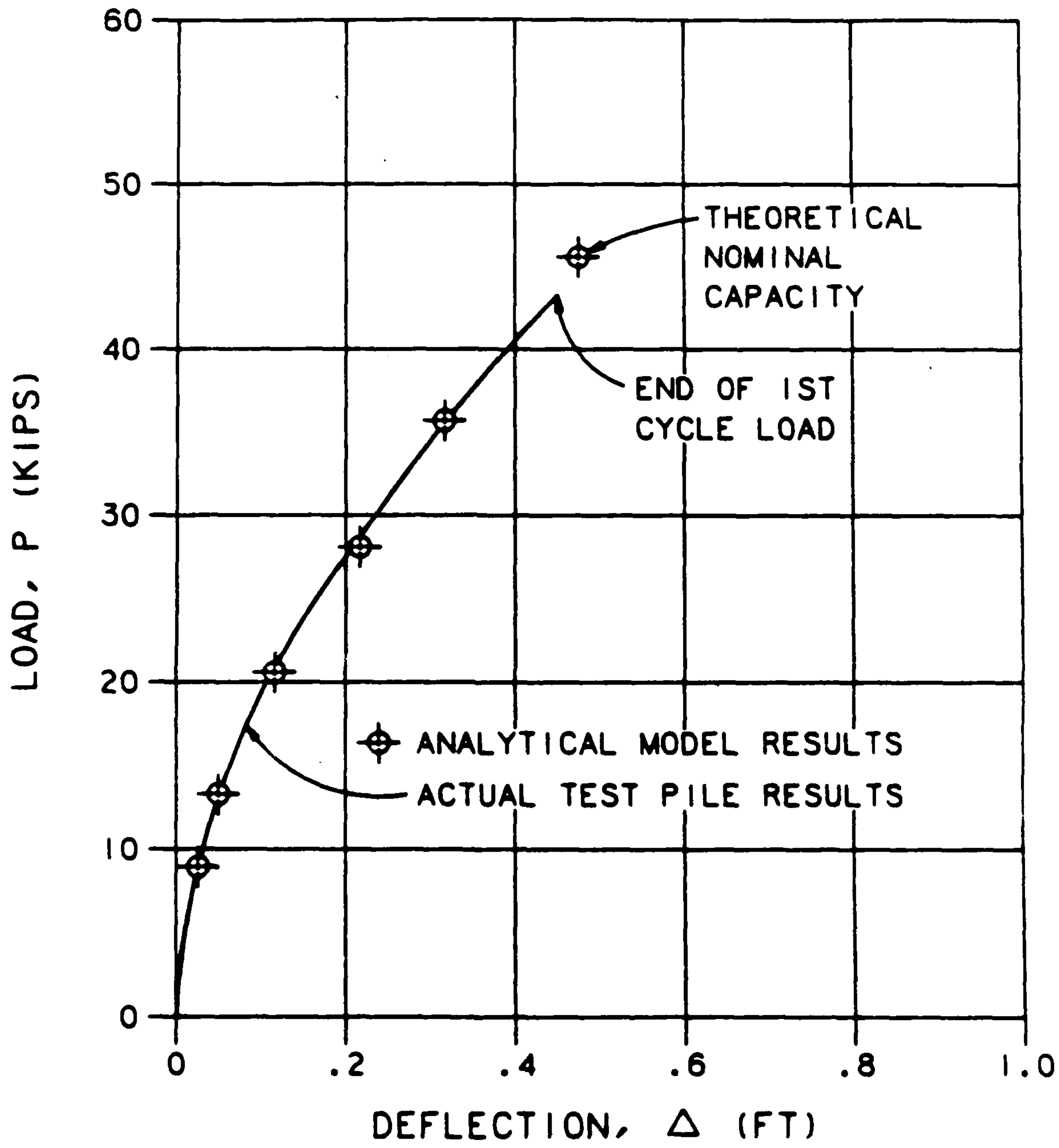


FIGURE 3.3, COMPARISON OF ANALYTICAL MODEL
WITH NCEL TEST PILE MK-19 RESULTS

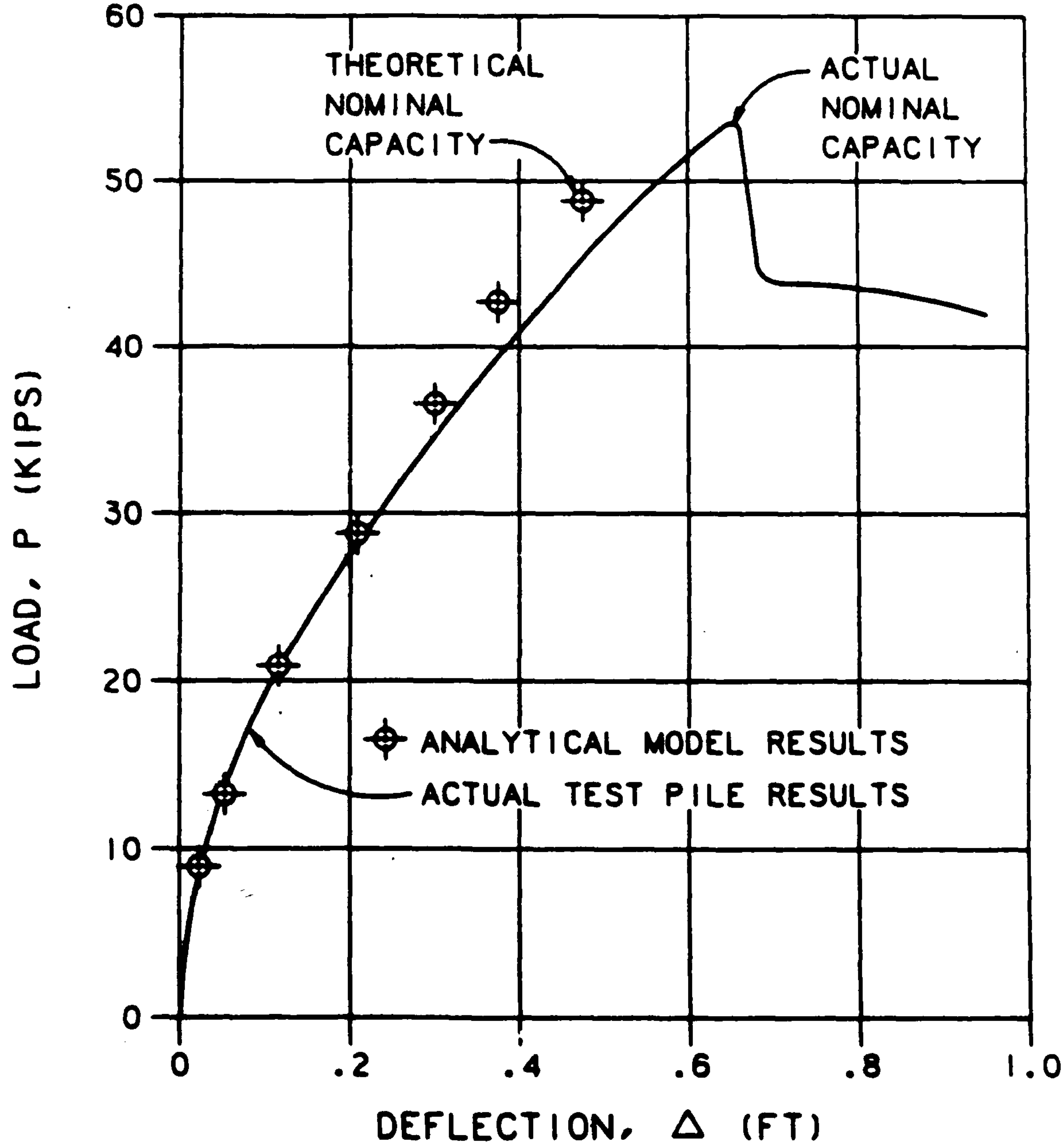


FIGURE 3.4, COMPARISON OF ANALYTICAL MODEL WITH NCEL TEST PILE MK-22 RESULTS

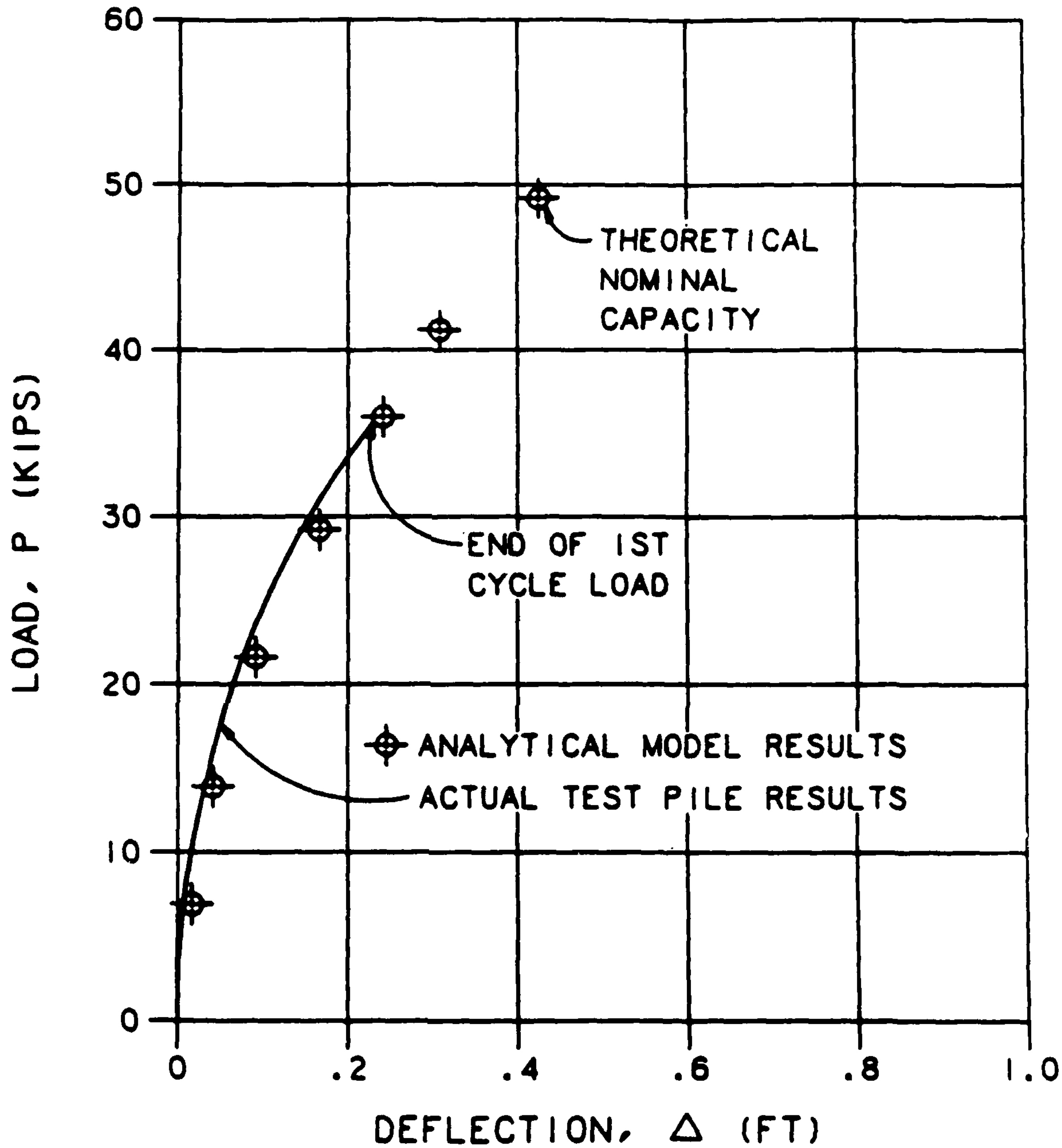


FIGURE 3.5, COMPARISON OF ANALYTICAL MODEL
WITH NCEL TEST PILE MK-28 RESULTS

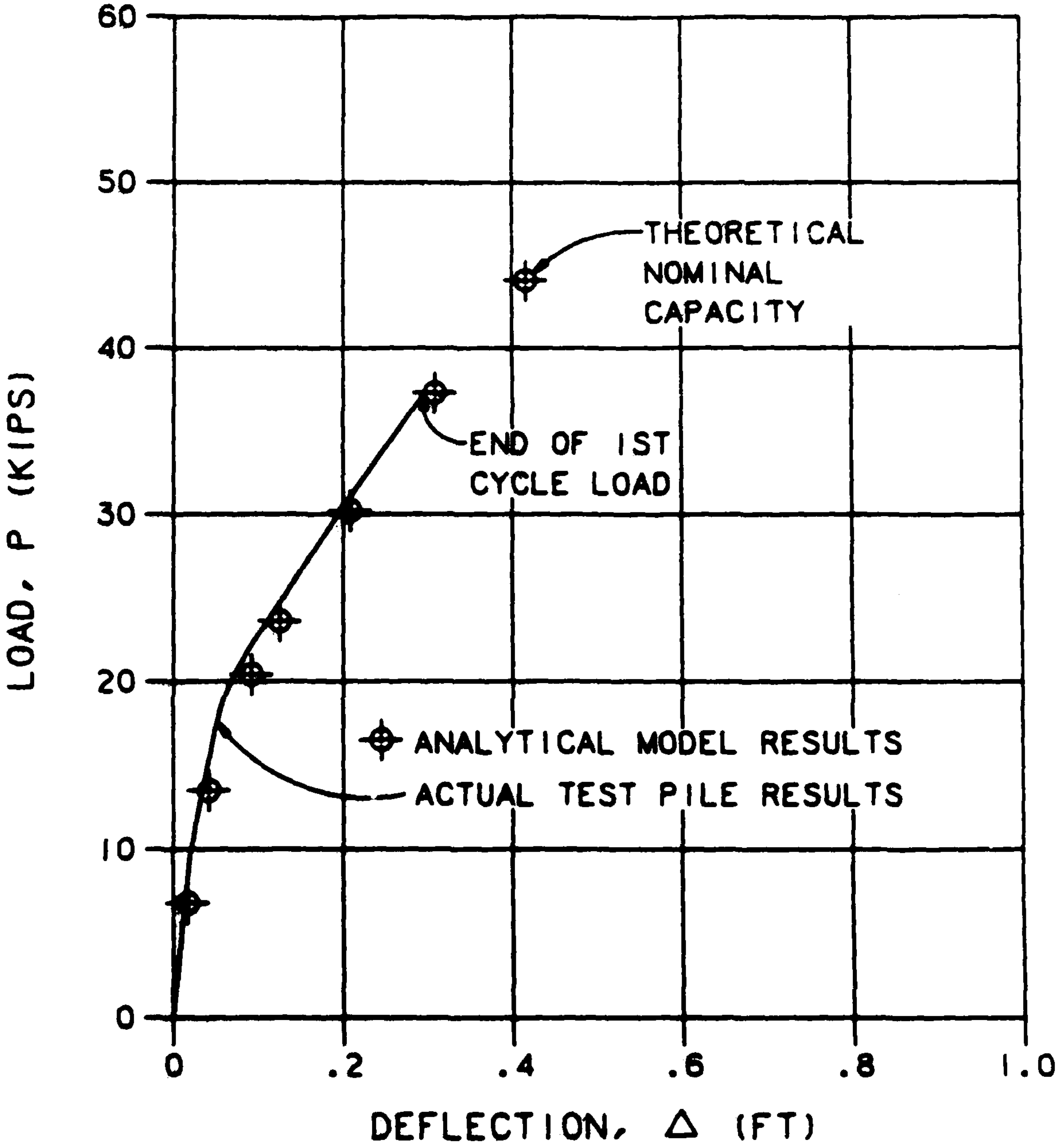


FIGURE 3.6, COMPARISON OF ANALYTICAL MODEL WITH NCEL TEST PILE MK-29 RESULTS

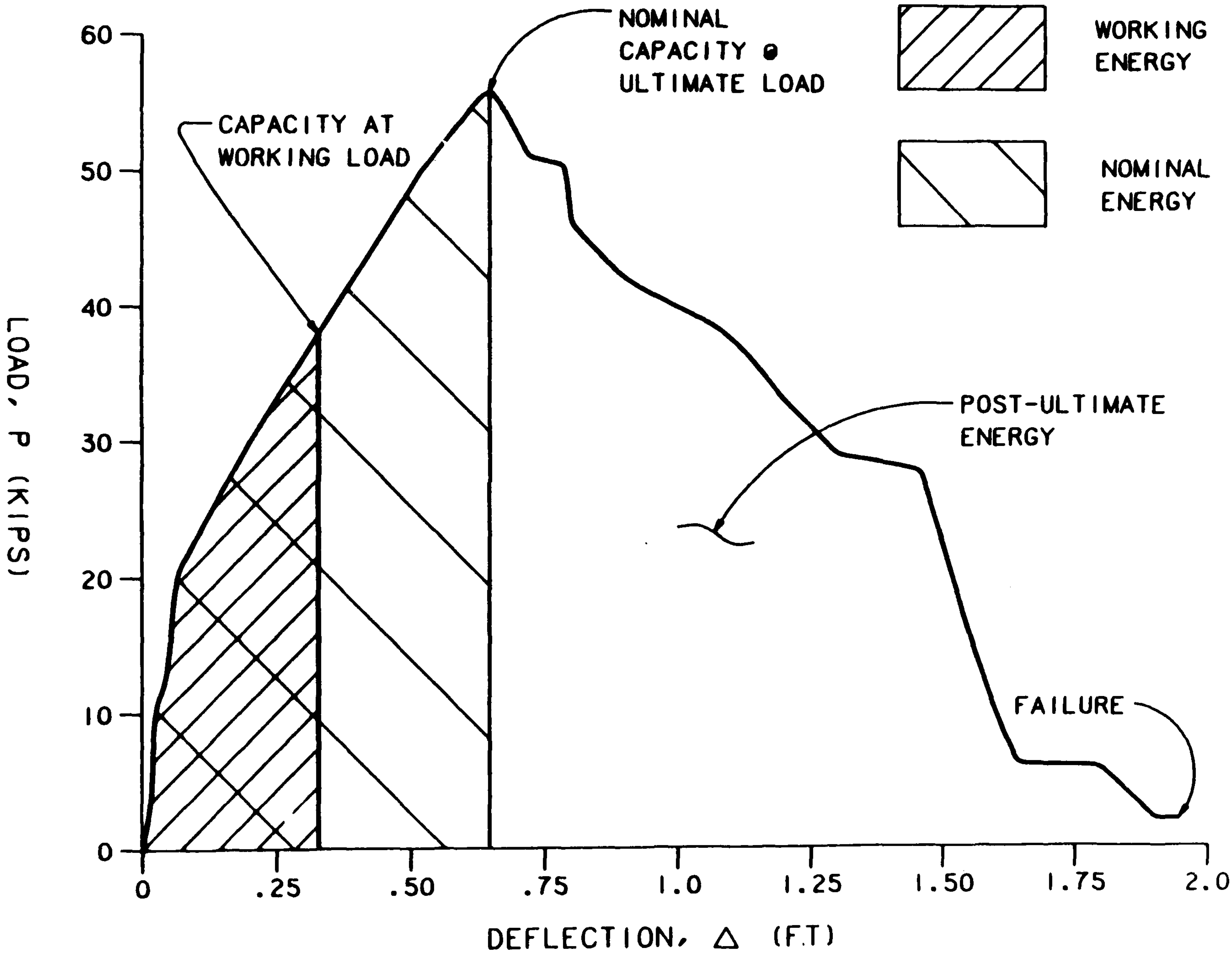


FIGURE 3.7, MONOTONIC LOADING vs DEFLECTION FOR NCEL TEST PILE MK-16

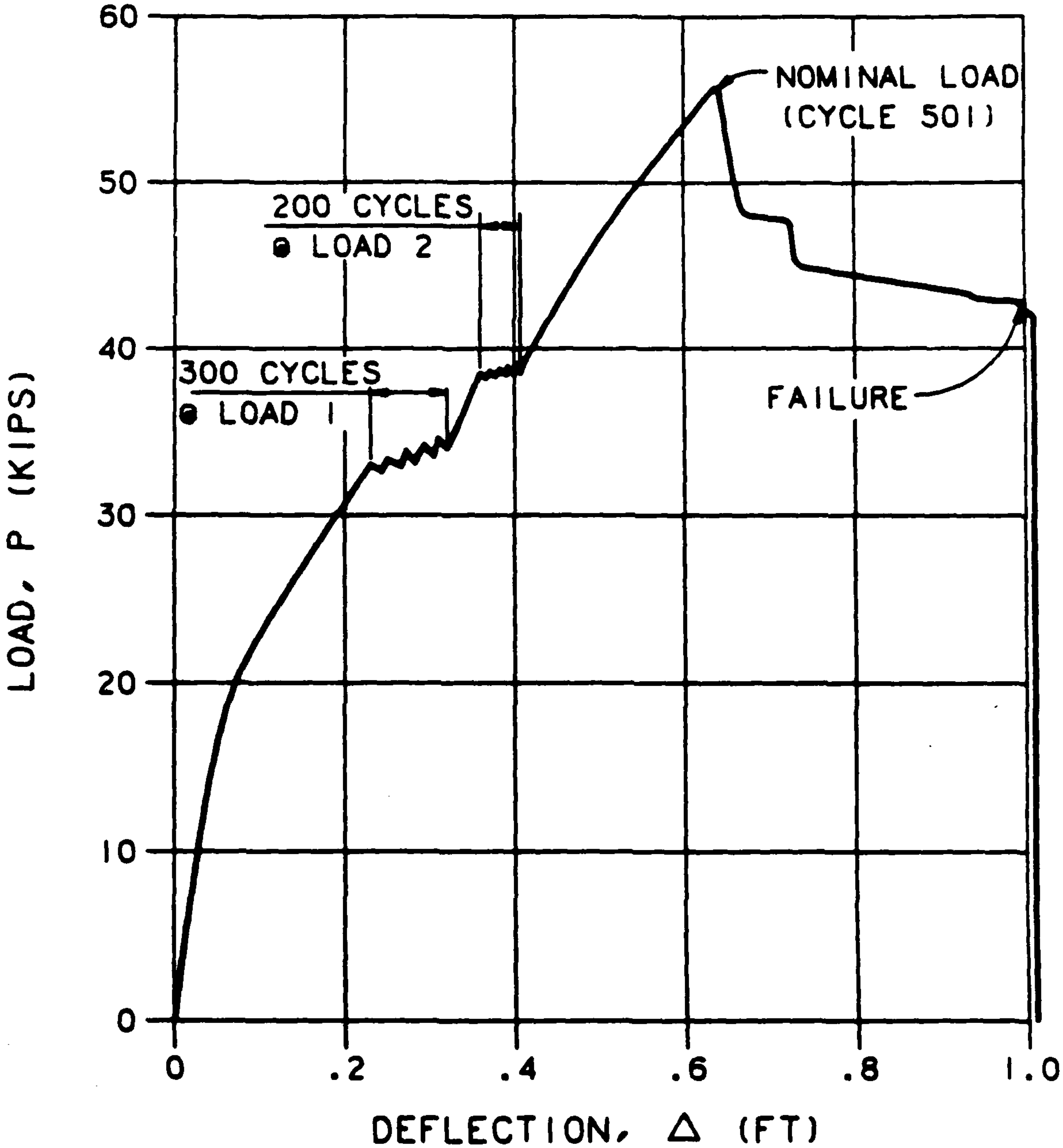


FIGURE 3.8, CYCLIC LOAD vs DEFLECTION FOR NCEL TEST PILE MK-12

SECTION 4
RECOMMENDED PILE CONFIGURATION

4.1 DESIGN CRITERIA

The design criteria are divided into two performance categories: normal and accidental berthing, as defined in Navy Design Manual DM 25.1 [4.1]. Working energy capacity (E_{wc}) represents the normal berthing performance category. The pile is expected to operate at this energy level for at least 125 cycles of load application without damage. Damage is defined as concrete spalling on the compression face or residual crack widths on the tension face greater than 0.012 in. It should be noted that most of the berthing impacts (and energies) during the pile's expected life will be significantly less.

Nominal energy capacity (E_{nc}) represents the accidental berthing performance category. The pile is expected to resist this accidental berthing impact but the designer should expect some damage to the pile. This may necessitate repairing or replacing the pile.

It should be noted that pile testing at NCEL has shown that the actual failure energy of the pile is significantly greater than the nominal energy capacity, hence providing a substantial additional reserve energy to failure.

The design criteria chosen for limiting the working energy in the prestressed concrete fender pile were based on both a rational approach and using the results from the NCEL test pile program. A discussion of these criteria follows:

a) Effective prestress (f_{pc}) of 600 psi. This value is based on the experience of the authors and NCEL laboratory testing. A minimum prestress is required to resist the handling and driving stresses and close small cracks that may open when the pile is loaded.

b) Maximum concrete compressive strain under working loads of 0.0021 in./in. The theoretical maximum compressive strain when the concrete crushes under monotonic loading was assumed to be 0.003 in./in. The compression side of the pile sustained damage when cycled at concrete strains greater than 0.0021 in./in. This limit should provide adequate performance for at least 125 cycles of load.

c) Maximum stress in strands at working energy (f_{pw}) of 210 ksi. On the tension side of the pile, there appears to be a correlation between the strand stress and the crack width in the pile.

d) The working energy (E_{wc}) must be no more than two thirds of the nominal pile energy (E_{nc}). This provides a minimum factor of safety of 1.5 in case of overload in the pile as required in DM 25.1 [4.1].

At the nominal energy, the concrete cover on the compression side may spall off.

The intended result of the above criteria is to limit the residual transverse crack width under cyclic loads to less than 0.012 in. over the life of the pile. Use of No. 3 rebar at 3-in. spacing for confinement steel is necessary to assure minimum crack widths.

4.2 RESIDUAL CRACK WIDTH

The residual crack width opening is apparently affected by the following:

a) Size of Stirrup: Larger diameter bars (No. 3) do a better job of confining the strands which are in a square or rectangular pattern. The No. 3 rebar performed significantly better than W5 wire. Smaller crack widths resulted from the use of No. 3 rebar than equivalent numbers of smooth small-diameter wire. Though the wires provided equivalent tensile strength, they were apparently not stiff enough to confine the strand without cross ties and strand slip occurred. The optimum diameter of the stirrup bar and the effects of cross ties with different bar diameters were not investigated in this study.

b) Type of Stirrup: It appears that the deformations on the rebar help prevent longitudinal cracks along the tension side strands from opening (or propagating); hence, the prestressing strand does not debond and the residual transverse cracks are smaller.

c) Stirrup Spacing: Large spacing results in more strand debonding, hence larger deflections and failure energies but also very large residual transverse crack widths and spalling of the concrete cover on the compression and tension faces.

d) Amount of Prestress: Crack width can be directly related to strand stress (strain). Therefore, it is desirable to provide the most prestressing strand at the lowest working stress that is economically possible.

Allowable residual crack widths based on measurements from the NCEL cyclic loading tests may be too severe. The location and magnitude of the loads in the test program were nearly constant. In service the load will vary in magnitude and location along the length of the pile and some of the cracks may close due to autogenous healing of the concrete, in water, after removal of the load. Hence, residual crack widths in service may be substantially smaller.

4.3 BASELINE PILE

Specifications for the baseline pile used in this phase of the work are as follows:

- 18-in.-sq pile
- Concrete strength: $f'_c = 8$ ksi
- Number of Stressing Units: Twenty 1/2-in.-diameter 270-ksi strands in a rectangular or circular pattern
- Effective concrete compressive stress: $f_{pc} = 600$ psi
- Confinement Reinforcing: No. 3 rebar at 3-in. spacing

4.4 PILE REINFORCING CONFIGURATIONS

The recommended strand configurations for each size pile are shown on the drawings in Section 8. The design aids presented in Section 5 are based on these combinations of pile size and strand layout. All the piles have a minimum 2-in. cover to the confining reinforcement. All the strands are 1/2-in.-diameter 270-ksi strand. They are placed so that there is a minimum 2-in. opening across the center of the pile to allow for a pipe sleeve to be placed every 2 ft to attach UHMW rubstrips.

The confining reinforcement shown for the rectangular strand pattern pile is No. 3 reinforcing bars. Smooth wire should not be used for this configuration because it is not as effective in limiting the crack widths and compression zone deterioration under cyclic loads. The confining reinforcement recommended for the circular strand pattern can be either No. 3 rebar or a smooth wire of equivalent area.

SECTION 5
DESIGN AIDS

5.1 PURPOSE OF THE DESIGN AIDS

This section of the report focuses on the design and selection of prestressed concrete fender piles, including the concrete cross section, concrete strength, and strand pattern to resist the design berthing energy for a particular pile design length. The design aids are intended to assist the design engineer in selecting the required pile configuration. It is assumed that the pile design span length (L_e) has been established by the design engineer for the specific site. Furthermore, the design engineer must establish the maximum design working energy (E_w) to be resisted by an individual pile based on an elastic distribution of the total ship berthing energy (E) applied to the fender system. The total berthing energy of the vessel and distribution to the piles can be calculated using DM 25.1 [4.1]. The distribution of the total energy (E) in the fender system to arrive at the minimum energy in an individual pile (E_w) is not part of this study.

5.2 THEORY AND DEVELOPMENT

5.2.1 Assumptions and Parameters

The design aids are based on the computer program FENDER. The design aids are valid for the following parameters and assumptions. If any of these parameters are not followed, it is the designer's responsibility to thoroughly examine the consequences.

a) Concrete

- o Normal weight concrete
- o Rectangular concrete cross section
- o Effective concrete stress, f_{pc} = 600 psi
- o Pinned support end conditions
- o Single point loading on the pile

b) Prestressing Steel

- o Stress relieved strand, or
- o Low relaxation strand
- o Specified tensile strength of strand, f_{pu} = 270 ksi

More detailed information on the computer program FENDER is discussed in previous reports (see Ref. 1.3).

As first mentioned in Section 4, there are three criteria that must be satisfied in order to determine the maximum allowable working energy (E_{wc}) in the pile section for a given length:

- o Maximum concrete compressive strain ≤ 0.0021 in./in.
- o Maximum stress in the prestressing steel ≤ 210 ksi
- o Nominal pile energy capacity (E_{nc}) must be at least 50% greater than the pile working energy capacity, $E_{nc} > 1.5 * E_{wc}$

5.2.2 Pile Energy versus Length

Several important conclusions can be drawn when studying the relationship between the pile energy and length. These directly impact the way the design aids are presented and are important to the design engineer in using the aids. The following relationships hold true for a given pile cross section and concrete strength with a specific reinforcing configuration.

a) The energy capacity of the pile is linearly proportional to the pile design length. The computer models used to generate data for the graphs and chart are based on a 65-ft design span. The design criteria that control the energy capacity for a specific pile cross section will control for all the possible pile span lengths. The amount of energy that the same pile cross section of different length could absorb is a direct ratio expressed as coefficients C_{ew} (at working energy capacity) and C_{en} (at nominal energy capacity).

See Tables 5.2 and 5.3. Notice that all the lines extend back linearly to the origin. See Figures 5.2 and 5.7.

b) For a specific design span length (say 65 ft), the allowable working energy in the pile is independent of where the load is applied along its length. Therefore, whether the pile is loaded 15 or 20 ft from the top support, the allowable working energy that the pile will absorb is the same. This is theoretically true for any length but, to limit the shear in the pile as well as provide an adequate bond length for the strand, the load should not be applied closer than 7 ft from the end support. If it is anticipated that the pile would be loaded closer than 7 ft from the top, the designer should not rely solely on the pile to absorb the berthing energy but should use rubber fenders (or some other method) in conjunction with the pile.

c) Although the allowable energy that a pile can absorb is independent of where the pile is loaded along its length, the corresponding deflection and magnitude of load do vary with the location of load. The pile energy is proportional to load times deflection. For a specific pile configuration and length, the allowable pile energy is equal to a coefficient times the product of the applied load and pile deflection, E = (coef) * P * Defl. This coefficient has been calculated to be approximately 0.66 (varying ±3%) for a range of pile sizes and reinforcing configurations. This coefficient will be used later in the report for the deflection calculation. If the pile cross section behaved perfectly elastically, the coefficient would be 0.5 for all cases. As stated previously, the analysis was performed on a 65-ft pile loaded 15 ft from the end. Knowing that the maximum working energy in the pile is independent of the location of the applied load, and that the coefficient in the equation above equals 0.66, one can see that the applied load and corresponding deflection in the pile are inversely proportional to one another as the location of load changes. Given the pile energy and applied load, deflection can be determined by the equation, Defl = 1.5 * E/P.

It is recommended that the fender piles have only sufficient penetration to resist the horizontal and vertical forces at the bottom of the pile but not so much that it creates moment fixity. The design span, L_e, of the pile is calculated as the length from the top support to the assumed location of the bottom support. If the pile fixity cannot be avoided due to the type of material the pile is driven into, it will decrease the energy absorption of the pile. It is conservative in this instance to use a pile span length (L_e) in the charts equal to the length from the top of the pile to the moment inflection point, the location of the inflection point having been calculated based on full pile fixity at the bottom. Similarly, the detail of the connection of the pile top to the pier structure should allow for rotation so as to not create any moment fixity.

5.3 GRAPHIC AIDS AND CHART

For a specific pile length and working energy requirement, the variables that a designer has to choose from to satisfy the design criteria are as follows:

a) Pile Size: 14-, 16-, or 18-in. sq sections.

Sizes smaller than 14 in. and larger than 18 in. were not investigated in this study.

b) Concrete Strength: f'_c = 6, 7, 8, 9, or 10 ksi.

Typically, concrete strengths ranging from 6 to 8 ksi were used in the charts. For the baseline case, 18-in. pile with 20 strands, higher strength concretes with f'_c = 9 and 10 ksi were also investigated.

c) Number of Strands: 12, 16, and 20.

The possible number of strands to be used in the pile cross section was arrived at several different ways. First, the strand pattern had to be symmetrical so that only one level of prestress would be required, with an equal number of strands on each side. Second, the pattern had to allow for a 2-in.-diameter pile sleeve to be inserted perpendicular to the strands every 2 ft to allow attachment of a rubstrip. This eliminated odd numbers of strands on each face of the pile.

For each individual pile size, there were certain design criteria that limited the number of strands in a section. No more than 16 strands would fit in the 14-in.-sq section without using three layers of steel on each face. For the 18-in. section, using 12 strands was considered but this resulted in an unbalanced condition where the stress level in the strand reached a maximum well before the concrete strain reached its limit. Additionally, this pile was less efficient than the smaller piles. Therefore, 16 strands were considered to be the minimum for the 18-in. cross section. A total of 24 strands were considered in the 18-in. cross section but, when the results were compared to those with 20 strands, there was no significant increase in energy to justify the added cost of more strands.

d) Reinforcing Configuration

Two configurations are presented, square and circular confinement. The square reinforcing pattern is presented first. The baseline pile in this study has a square reinforcing configuration.

The design aid charts provide the pile energy versus pile design span length for different combinations of the variables presented above. Figures 5.2 and 5.7 give an overview of the ranges of energy and length.

The lines are all linear and all pass through the origin. For design purposes, the limits of the span length presented in Figures 5.3 and 5.8 are 40 to 90 ft. Lengths not in this range may be extrapolated from the data. Figures 5.3 and 5.8 give all the possible combinations and are intended to give the designer a first cut at the pile size required. Figures 5.4 through 5.6 and 5.9 through 5.11 present the same data for each pile size separately. Each line represents the "upper bound" of the energy available for a given pile configuration.

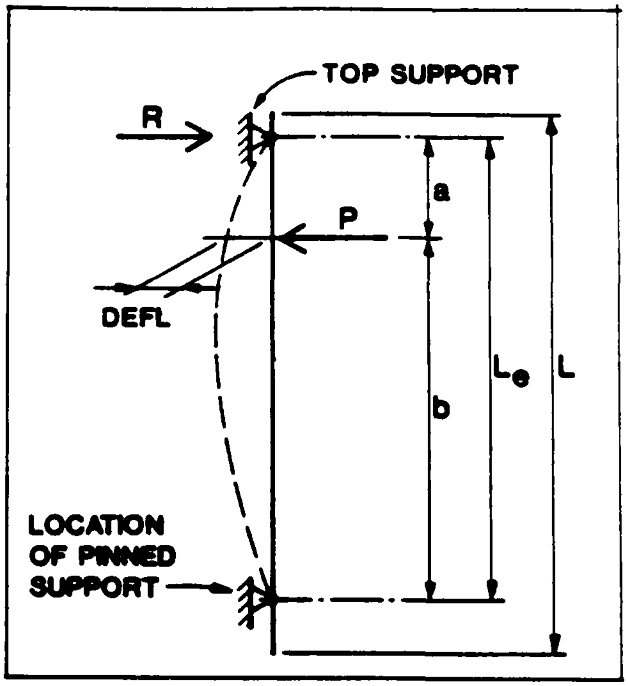
The energy for a given length pile may be the same for two or more combinations of concrete strength and number of strands. This reflects the sensitivity of the pile to changes in the design variables. Some rounding off of values was necessary to simplify the graphs. The design values shown are typically within ±3% of the computed value after rounding off.

After a pile configuration has been chosen based on the energy requirements and the pile span length, the designer should check the load in the pile and the corresponding deflection. Tables 5.2 and 5.3 provide design information for each pile configuration at two specific energy levels, at working energy and at nominal capacity. For a given pile size, concrete strength, and reinforcing configuration, the designer can determine the maximum energy in the pile by selecting coefficients for the working energy capacity (C_{ew}) and the nominal energy capacity (C_{en}). To calculate the energy, multiply the coefficient by the pile span length (L_e).

For a given pile size, concrete strength, and reinforcing configuration, the designer can select the moment in the pile at the point of loading for the working energy capacity (M_{wc}) and the nominal energy capacity (M_{nc}). Knowing the moment in the pile, the applied load that produces that moment can be found by statics. Based on the energy in the pile and the calculated applied load, the deflection at the location of applied load can be determined. This deflection is not necessarily the maximum deflection that occurs along the length of the pile.

Note that a distinction is made between the anticipated maximum energy the ship will impart to a fender pile (E_w) and the actual working energy capacity that a given pile configuration can absorb (E_{wc}). The capacity (E_{wc}) should always be equal to or greater than the actual energy (E_w). When calculating the fender reactions and the pile deflections, it is conservative to use properties based on the pile energy capacity (E_{wc}).

5.4 DESIGN PROCEDURE OUTLINE



Step 1: Determine basic parameters.

a) Pile design span length, L_e (ft)

b) Load location,
"a" (ft) and "b" (ft)

c) Maximum working energy in pile,
E_w (ft-k)

Step 2: Knowing L_e and E_w,

a) Select pile size (in. x in.) from Figure 5.3 or 5.8

b) Select concrete strength and number of strands from Figures 5.4 through 5.6 or 5.9 through 5.11

Step 3: Knowing pile configuration, use Table 5.2 or 5.3 to find C_{ew}, C_{en}, M_{wc}, and M_{nc}. Using these constants,

a) Calculate maximum pile energy capacities based on L_e

$$E_{wc} = C_{ew} \times L_e \text{ (ft-k)}$$
$$E_{nc} = C_{en} \times L_e \text{ (ft-k)}$$

b) Calculate range of applied pile loads based on pile capacities. Distances "a" and "b" vary, depending on water elevation.

$$P_{wc} = M_{wc} \times L_e/(a \times b) \text{ (kips)}$$
$$P_{nc} = M_{nc} \times L_e/(a \times b) \text{ (kips)}$$

Note: The load P is maximum when the dimension "a" is a minimum.
The load P is minimum when the dimension "a" is a maximum.
$a \geq 7$ ft

Step 4: Calculate range of deflections.

$$Defl_{wc} = 1.5\ (E_{wc}/P_{wc}) \text{ (ft)}$$
$$Defl_{nc} = 1.5\ (E_{nc}/P_{nc}) \text{ (ft)}$$

Note: The deflection (Defl) is maximum when the load P is minimum.

Step 5: Calculate reaction of fender pile at top support.

$$R_{wc} = P_{wc} \times b/L_e$$

$$R_{nc} = P_{nc} \times b/L_e$$

Note: The reaction R is a maximum when the load P is a maximum.

Step 6: If a rubber fender is used at the top of the pile, size it based on R_{wc}.

5.5 DESIGN EXAMPLE

A design example appears at the end of this section.

TABLE 5.1
SUMMARY OF DESIGN AID FIGURES AND TABLES

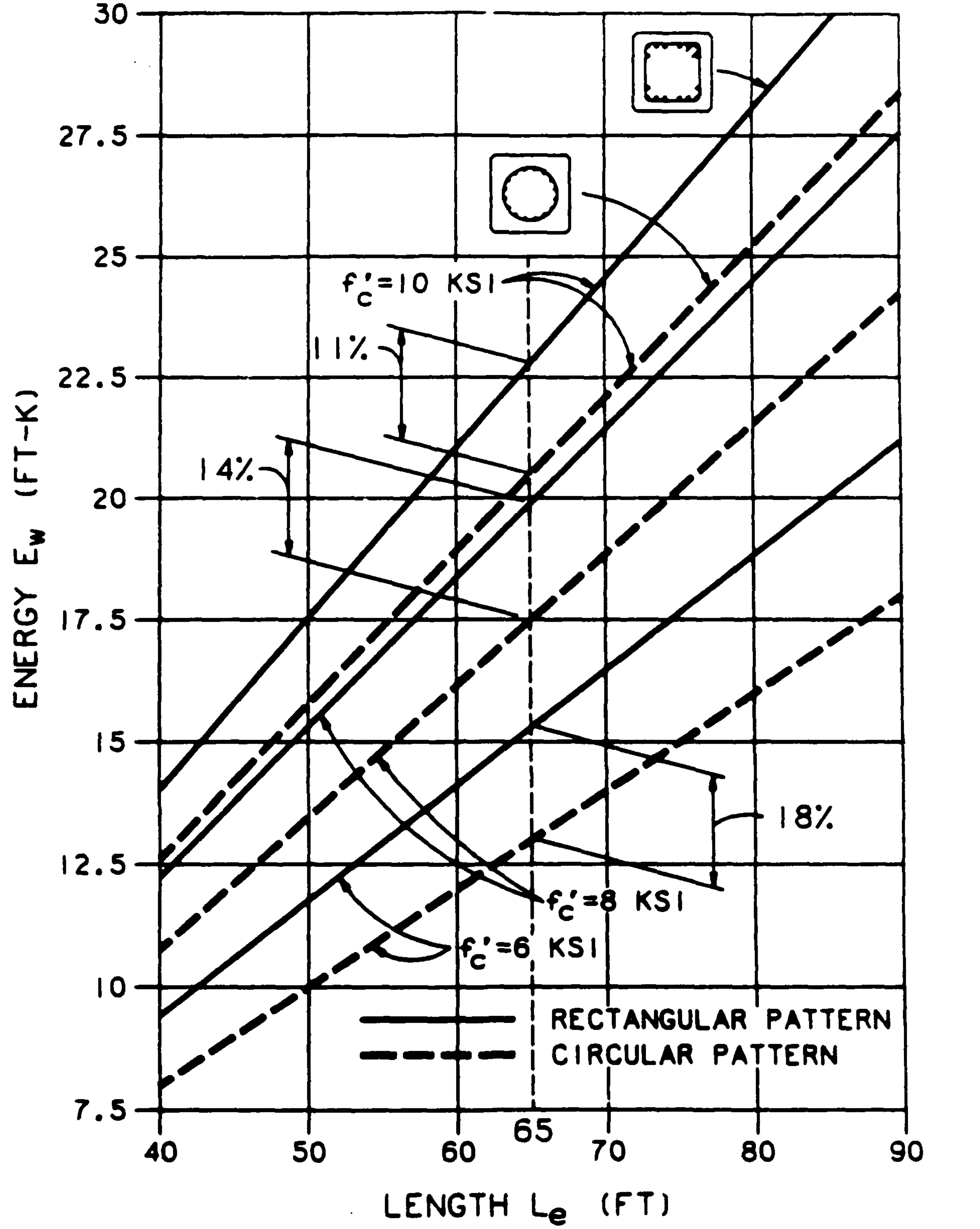
	Pile Size (in.)			Rectangular Strand Pattern	Circular Strand Pattern
	14	16	18		
Preliminary Size Based on E_w vs L_e	X	X	X	Figure 5.3	Figure 5.8
E_w vs L_e	X			Figure 5.4	Figure 5.9
E_w vs L_e		X		Figure 5.5	Figure 5.10
E_w vs L_e			X	Figure 5.6	Figure 5.11
Constants C and M	X	X	X	Table 5.2	Table 5.3

TABLE 5.2
CONSTANTS FOR PRESTRESSED CONCRETE
FENDER PILES WITH RECTANGULAR STRAND PATTERN

Pile Configuration			C_{ew}	C_{en}	M_{wc}	M_{nc}
			$= E_w/L_e$	$= E_n/L_e$		
Size	f'_c	No. of Strands	(ft-k/ft)	(ft-k/ft)	(k-ft)	(k-ft)
14" sq	6	12	0.128	0.211	124	146
	6	16	0.128	0.211	130	153
	7	12	0.148	0.245	133	158
	7	16	0.148	0.245	139	167
	8	12	0.168	0.275	140	167
	8	16	0.168	0.275	148	179
16" sq	6	12	0.183	0.275	190	216
	6	16	0.183	0.305	200	239
	6	20	0.183	0.305	208	250
	7	12	0.194	0.305	196	226
	7	16	0.211	0.352	214	256
	7	20	0.211	0.352	224	271
	8	12	0.194	0.323	198	233
	8	16	0.235	0.389	226	270
	8	20	0.235	0.400	237	290
18" sq	6	16	0.235	0.362	273	315
	6	20	0.235	0.389	286	343
	7	16	0.254	0.400	284	330
	7	20	0.271	0.445	307	367
	8	16	0.254	0.428	289	342
	8	20	0.306	0.491	325	386
	9	20	0.337	0.529	340	402
	10	20	0.351	0.562	354	414

TABLE 5.3
CONSTANTS FOR PRESTRESSED CONCRETE
FENDER PILES WITH CIRCULAR STRAND PATTERN

Pile Configuration			C_{ew} $= E_w/L_e$	C_{en} $= E_n/L_e$	M_{wc}	M_{nc}
Size	f'_c	No. of Strands	(ft-k/ft)	(ft-k/ft)	(k-ft)	(k-ft)
14" sq	6	12	0.117	0.189	113	132
	6	16	0.117	0.189	123	143
	7	12	0.137	0.223	122	143
	7	16	0.137	0.223	132	156
	8	12	0.155	0.254	129	153
	8	16	0.155	0.254	140	168
16" sq	6	12	0.155	0.235	162	184
	6	16	0.155	0.255	175	205
	6	20	0.162	0.266	187	220
	7	12	0.162	0.266	169	197
	7	16	0.185	0.300	188	222
	7	20	0.185	0.308	201	239
	8	12	0.171	0.300	173	207
	8	16	0.209	0.337	199	236
	8	20	0.209	0.352	213	257
18" sq	6	16	0.200	0.311	238	274
	6	20	0.200	0.334	254	299
	7	16	0.214	0.355	249	292
	7	20	0.237	0.388	273	323
	8	16	0.222	0.395	255	307
	8	20	0.269	0.434	289	343
	9	20	0.298	0.477	303	359
	10	20	0.315	0.517	311	374



NUMBER OF (1/2"Ø, 270 KSI) STRANDS = 20 FOR ALL CASES

SCALE: HORIZ 1" = 10 FT
VERT 1" = 3 FT-K

FIGURE 5.1, COMPARISON OF E_w vs L_e BETWEEN RECTANGULAR & CIRCULAR STRAND PATTERNS FOR 18" PILE

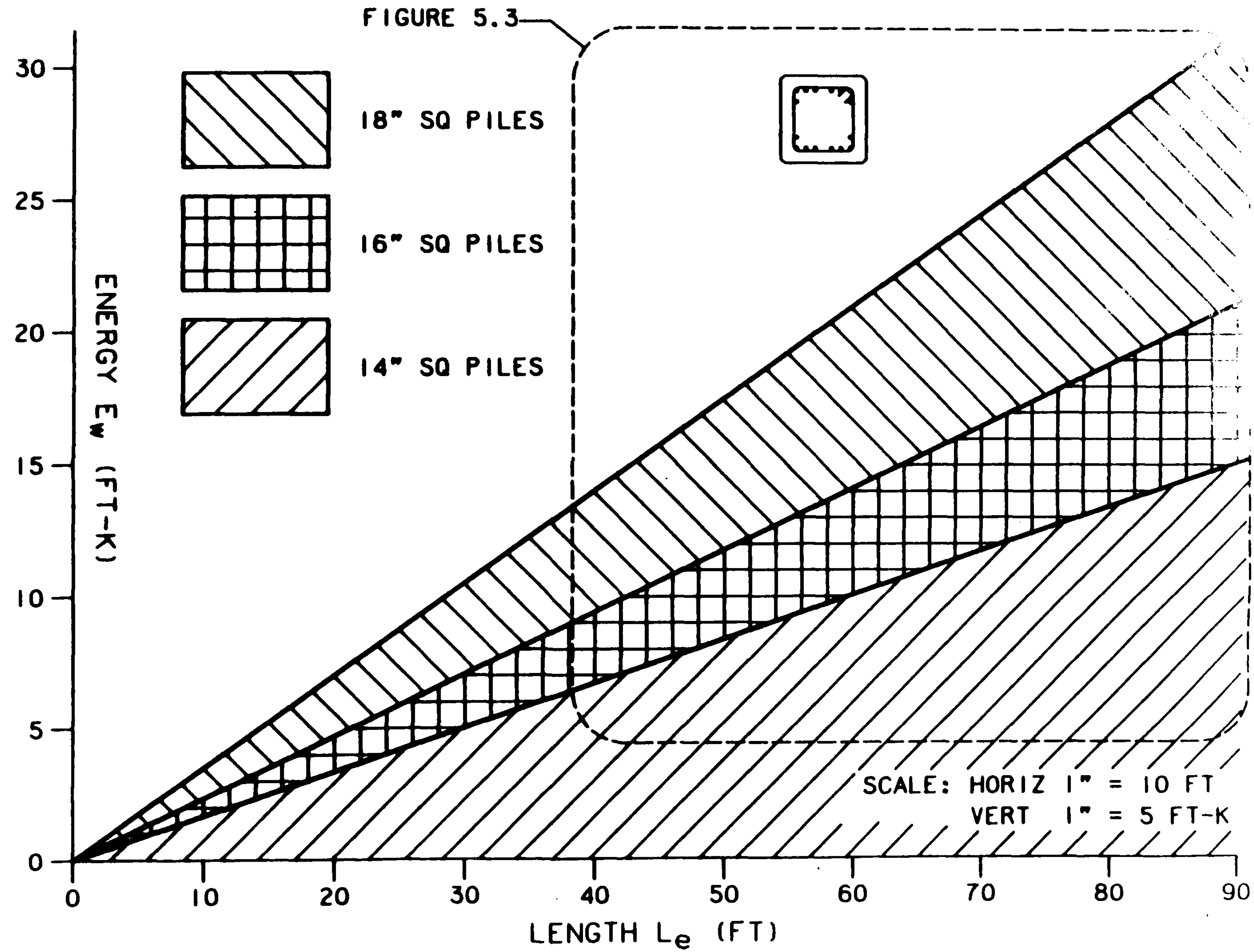


FIGURE 5.2, E_w vs L_e FOR RECTANGULAR STRAND PATTERN

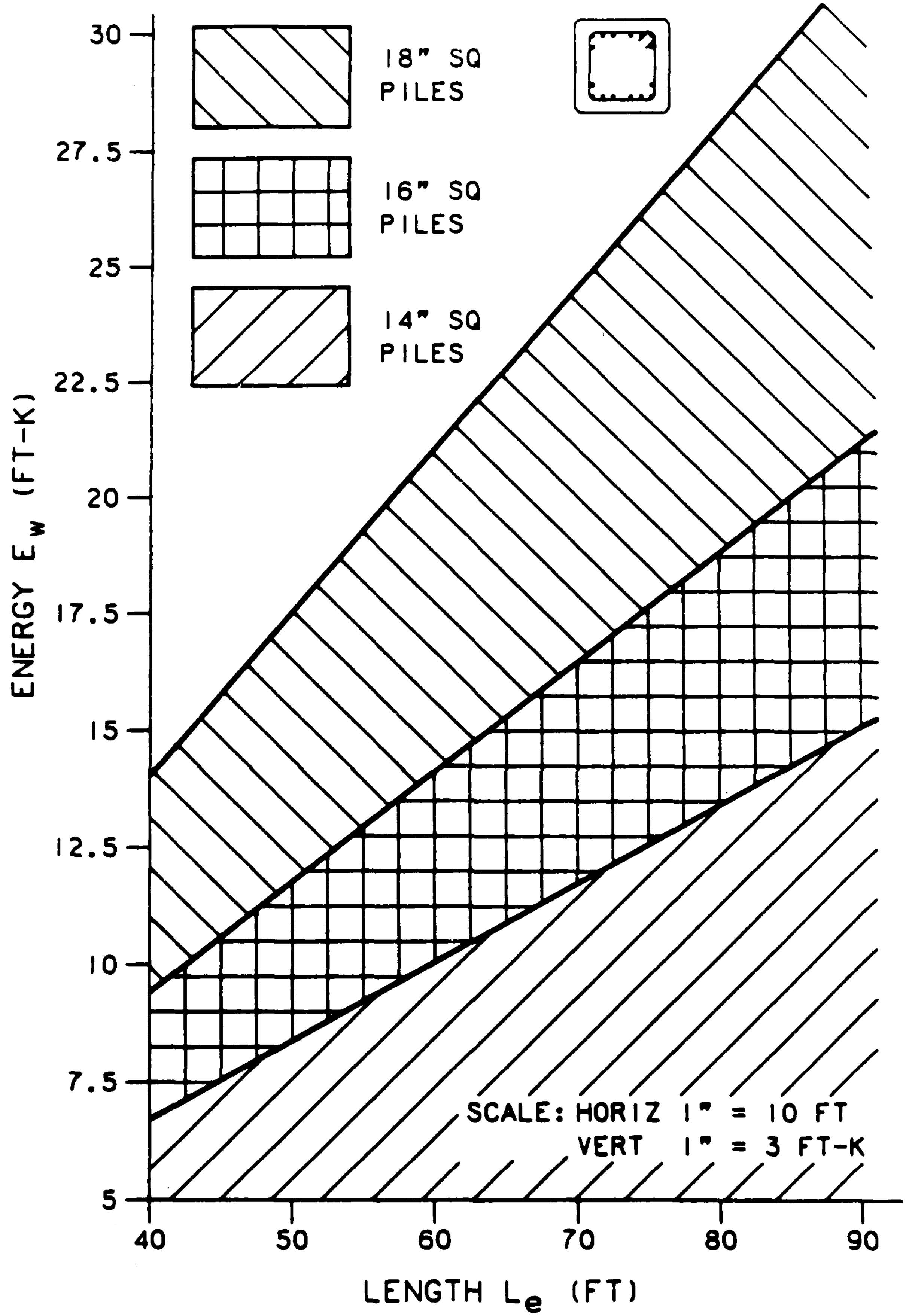
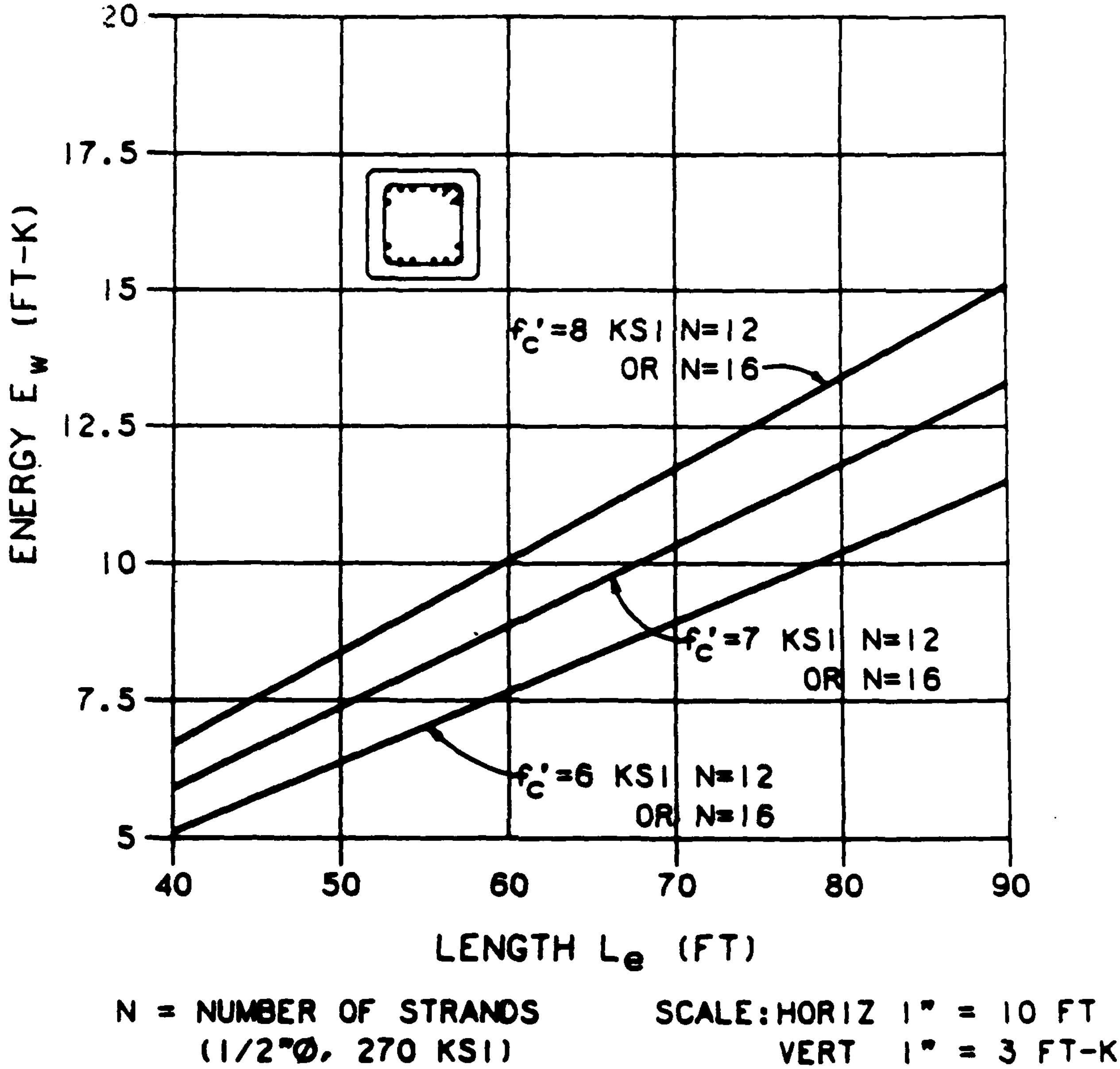
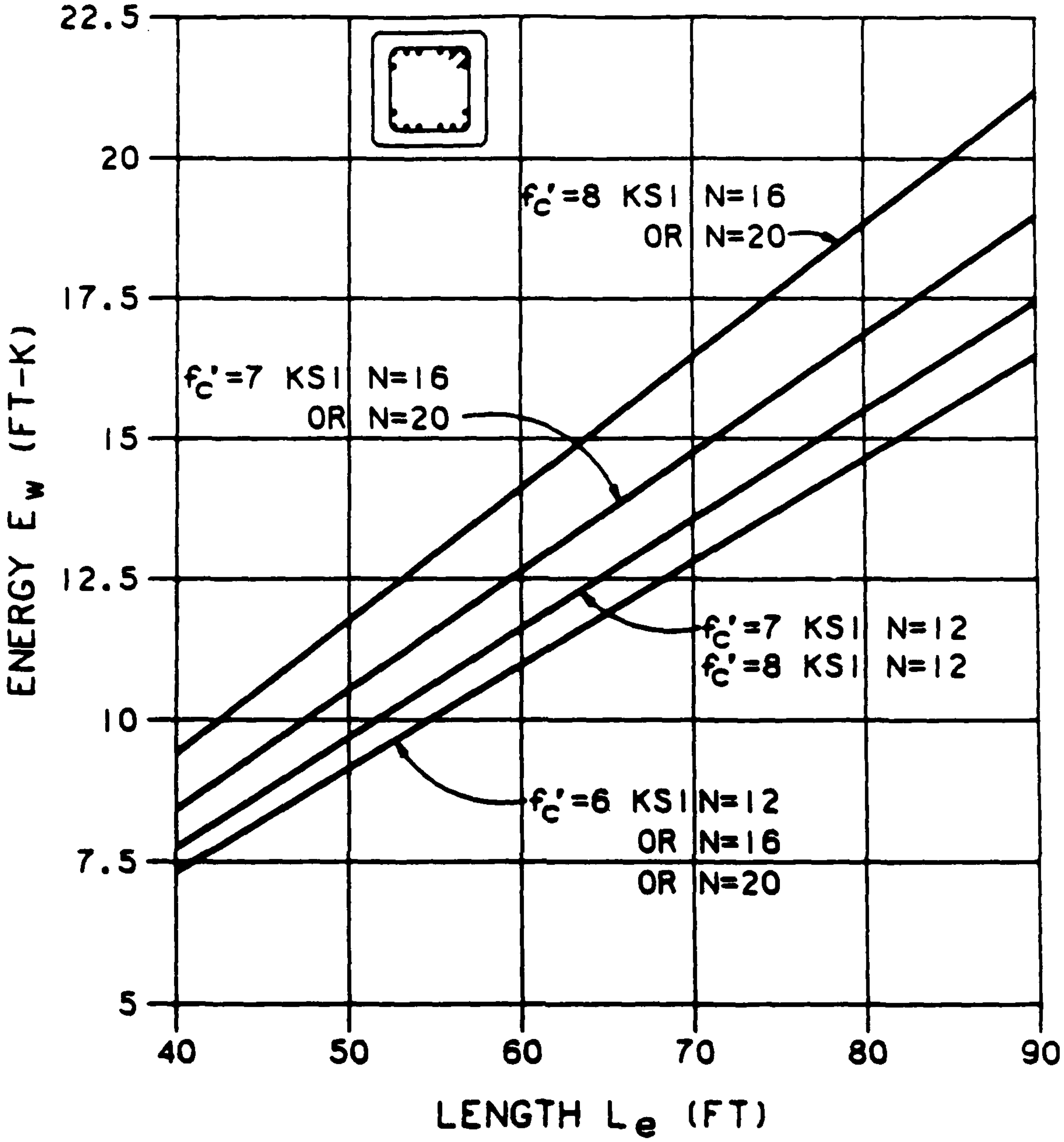


FIGURE 5.3, E_w vs L_e FOR 14", 16" & 18" PILES
W/ RECTANGULAR STRAND PATTERNS



14" X 14" PILE
RECTANGULAR PATTERN

FIGURE 5.4, E_w vs L_e FOR 14" PILE
W/ RECTANGULAR STRAND PATTERN

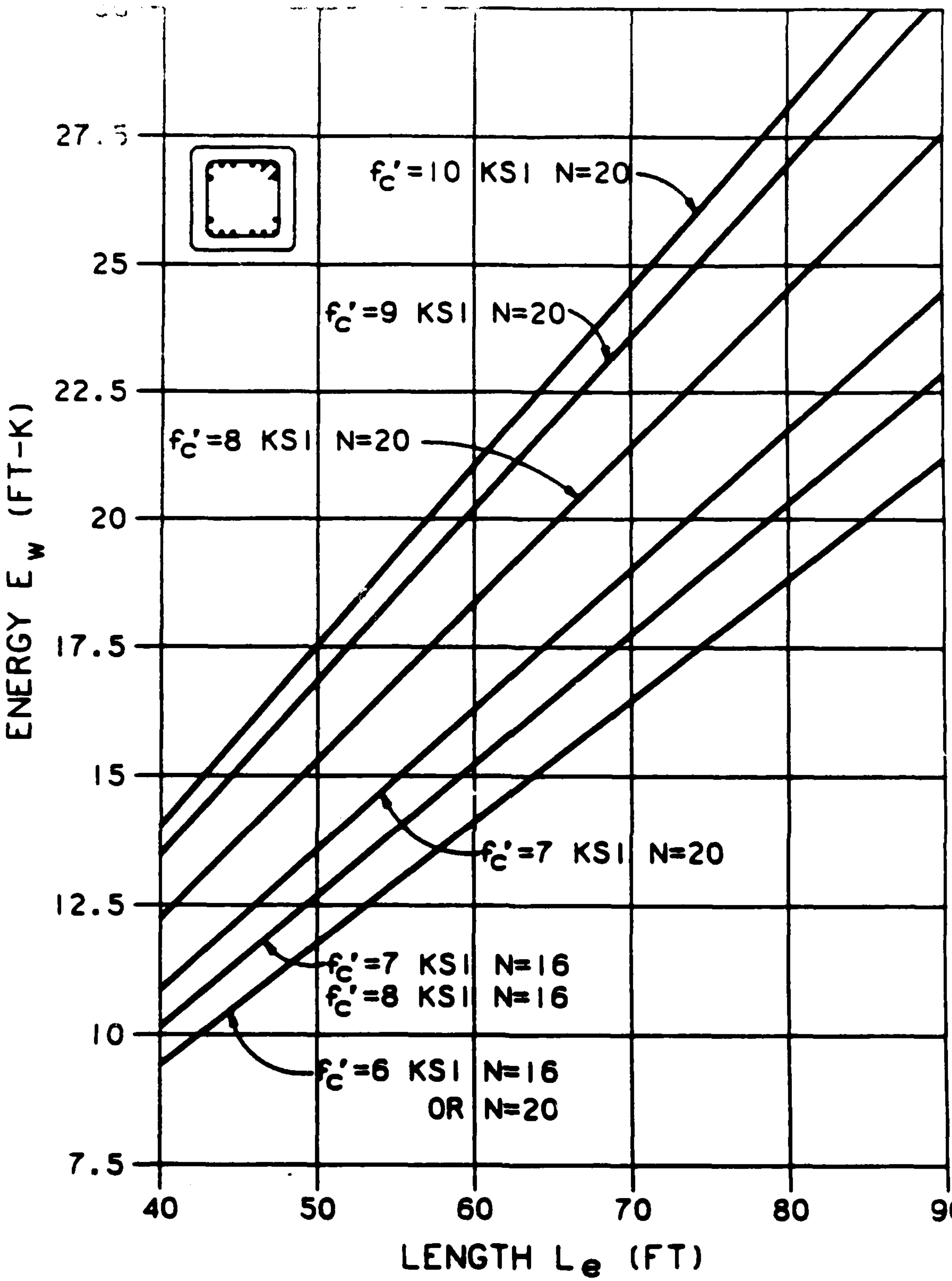


N = NUMBER OF STRANDS
(1/2"Ø, 270 KSI)

SCALE: HORIZ 1" = 10 FT
VERT 1" = 3 FT-K

16" X 16" PILE
(RECTANGULAR PATTERN)

FIGURE 5.5, E_w vs L_e FOR 16" PILE
W/ RECTANGULAR STRAND PATTERN



N = NUMBER OF STRANDS (1/2"Ø, 270 KSI)

SCALE: HORIZ 1" = 10 FT
VERT 1" = 3 FT-K

18" X 18" PILE
(RECTANGULAR PATTERN)

FIGURE 5.6, E_w vs L_e FOR 18" PILE W/ RECTANGULAR STRAND PATTERN

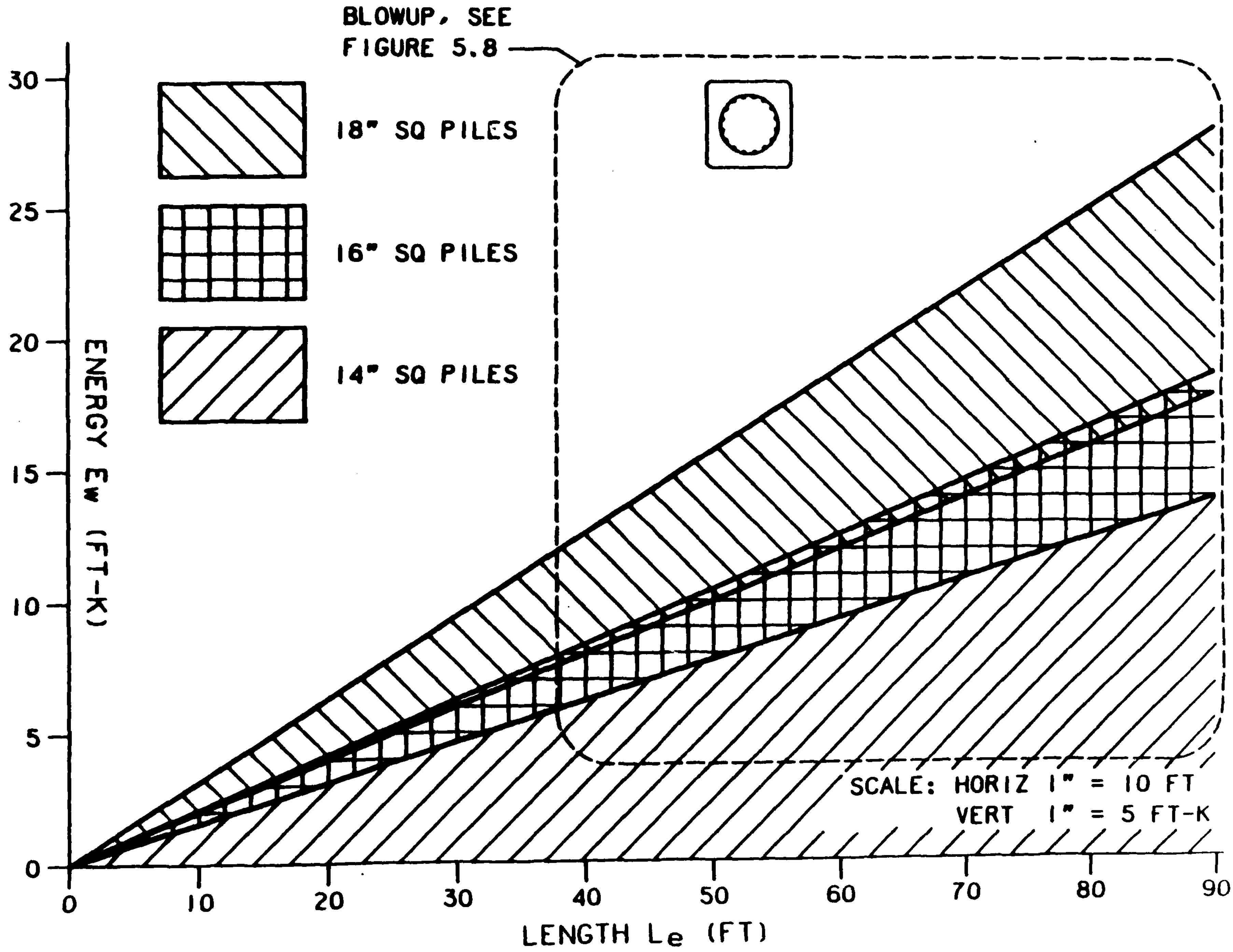


FIGURE 5.7, E_w vs L_e FOR CIRCULAR STRAND PATTERN

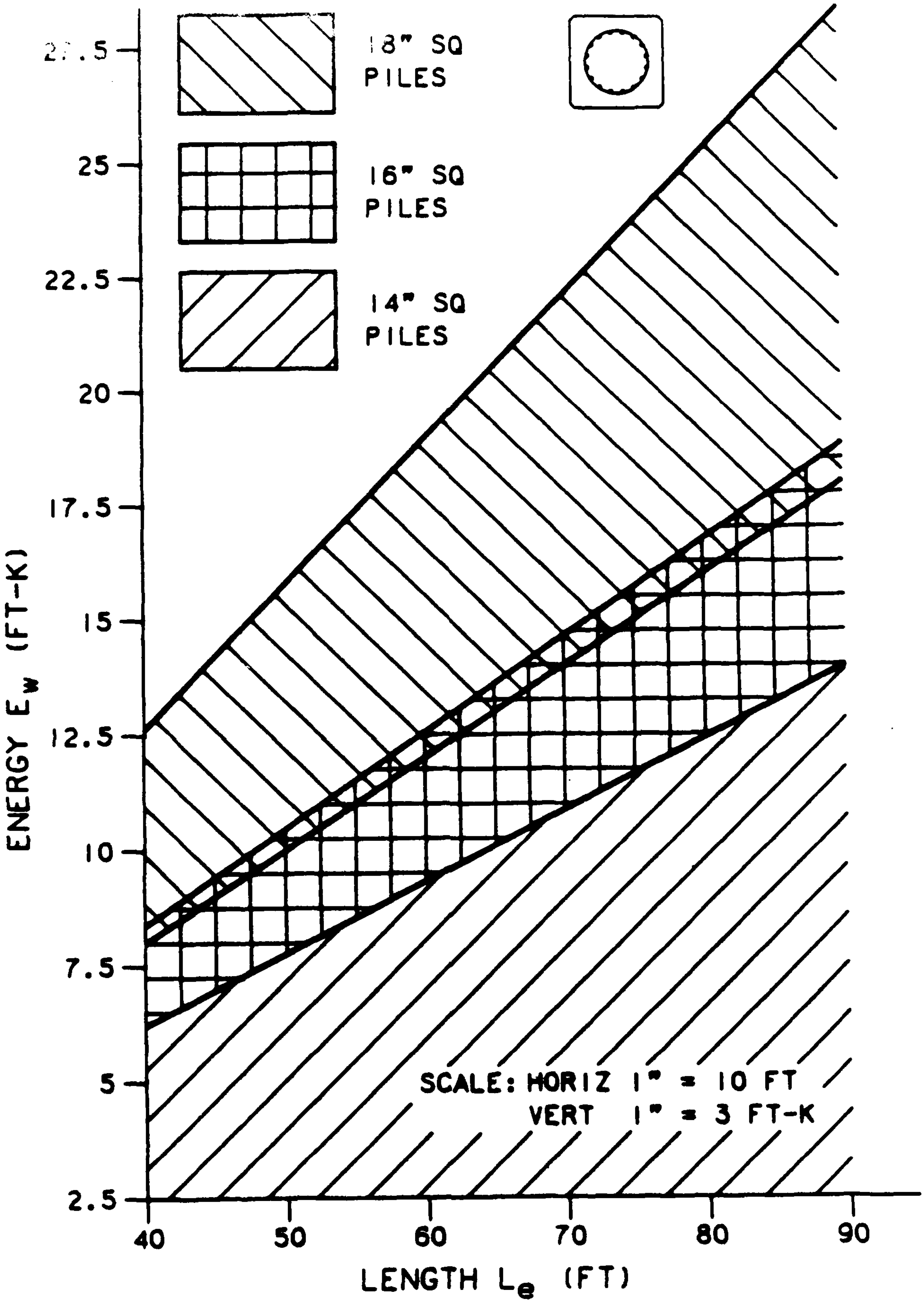
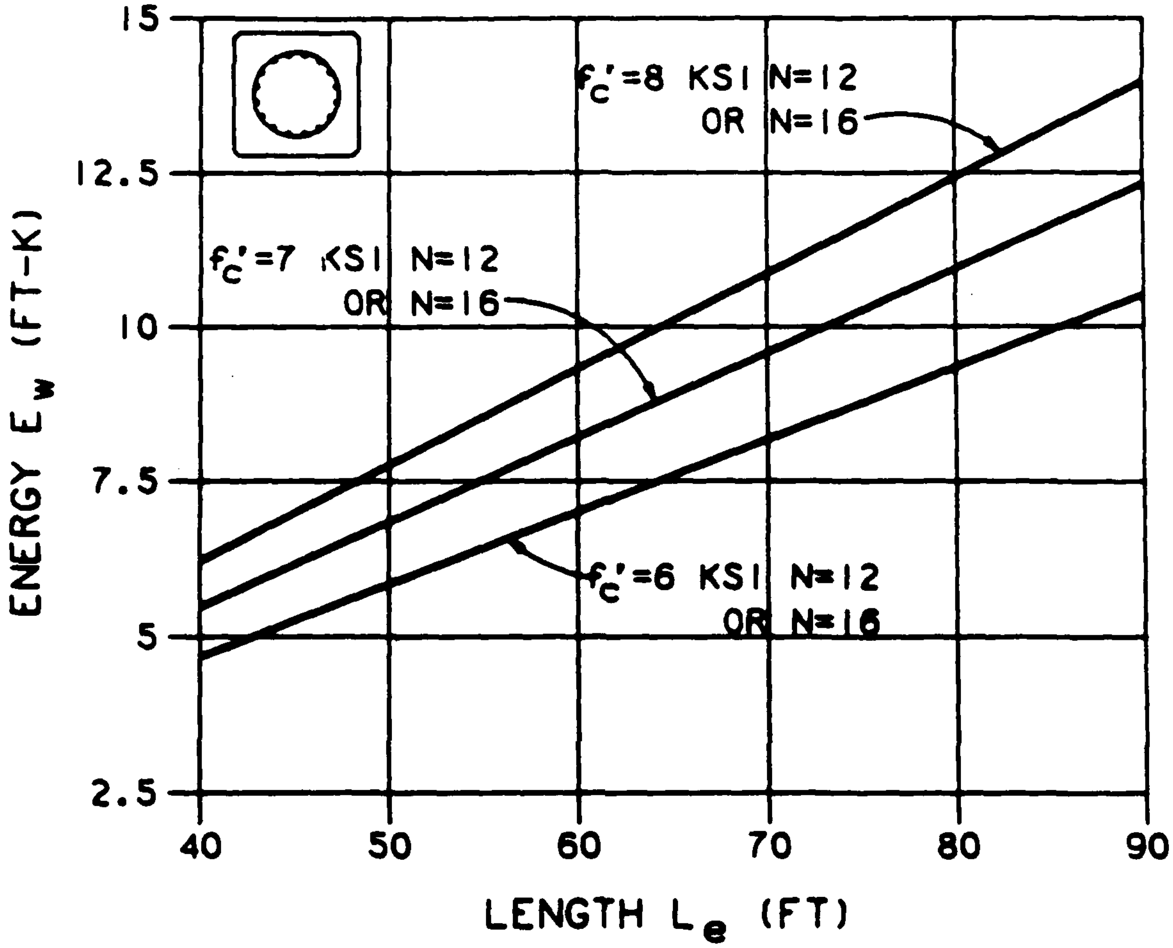


FIGURE 5.8, E_w vs L_e FOR 14", 16" & 18" PILES
W/ CIRCULAR STRAND PATTERN

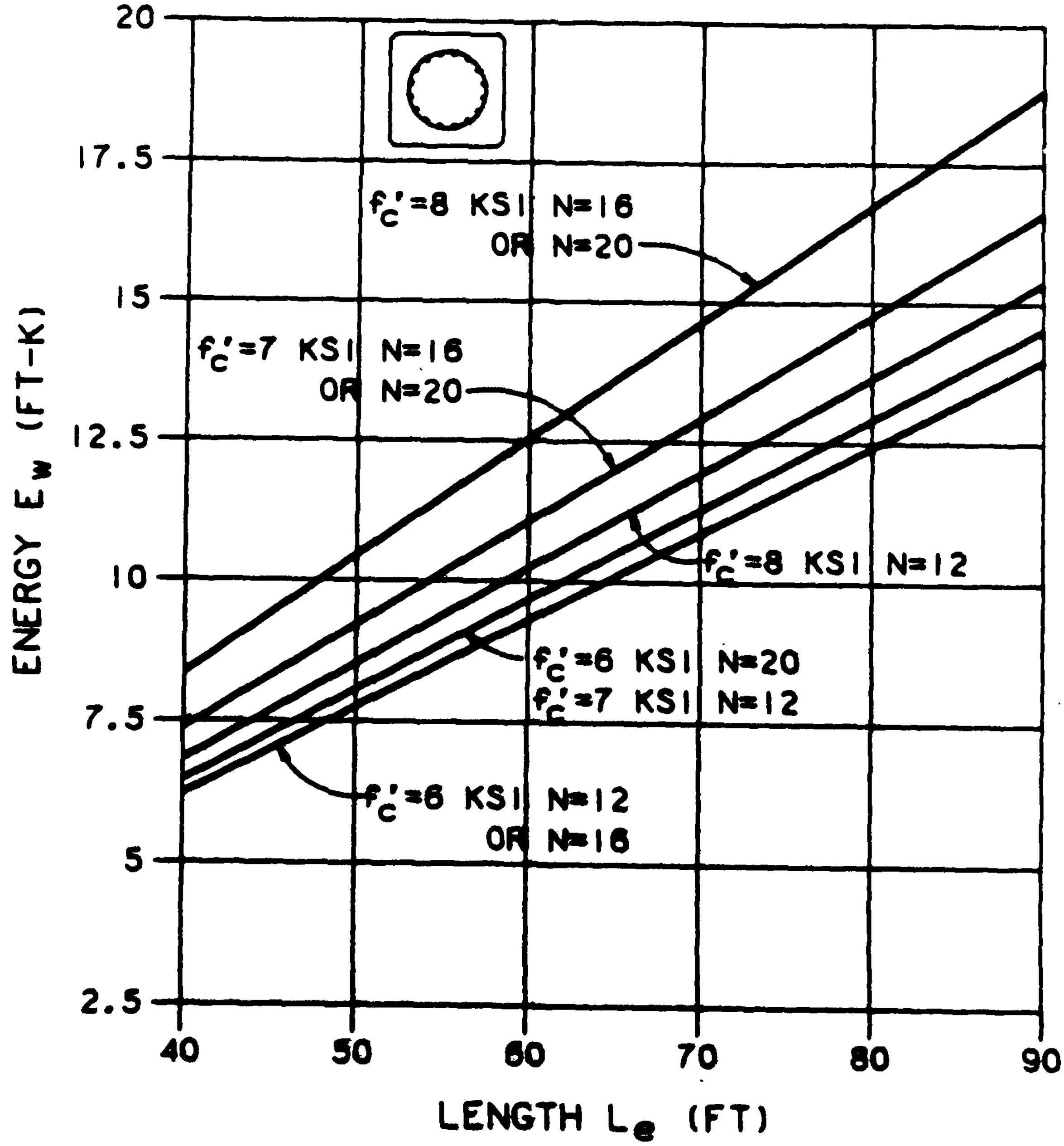


N = NUMBER OF STRANDS (1/2"Ø, 270 KSI)

SCALE: HORIZ 1" = 10 FT
VERT 1" = 3 FT-K

14" X 14" PILE
(CIRCULAR PATTERN)

FIGURE 5.9, E_w vs L_e FOR 14" PILE W/ CIRCULAR STRAND PATTERN



N = NUMBER OF STRANDS
(1/2"Ø, 270 KSI)

SCALE: HORIZ 1" = 10 FT
VERT 1" = 3 FT-K

16" X 16" PILE
(CIRCULAR PATTERN)

FIGURE 5.10, E_w vs L_e FOR 14" PILE
W/ CIRCULAR STRAND PATTERN

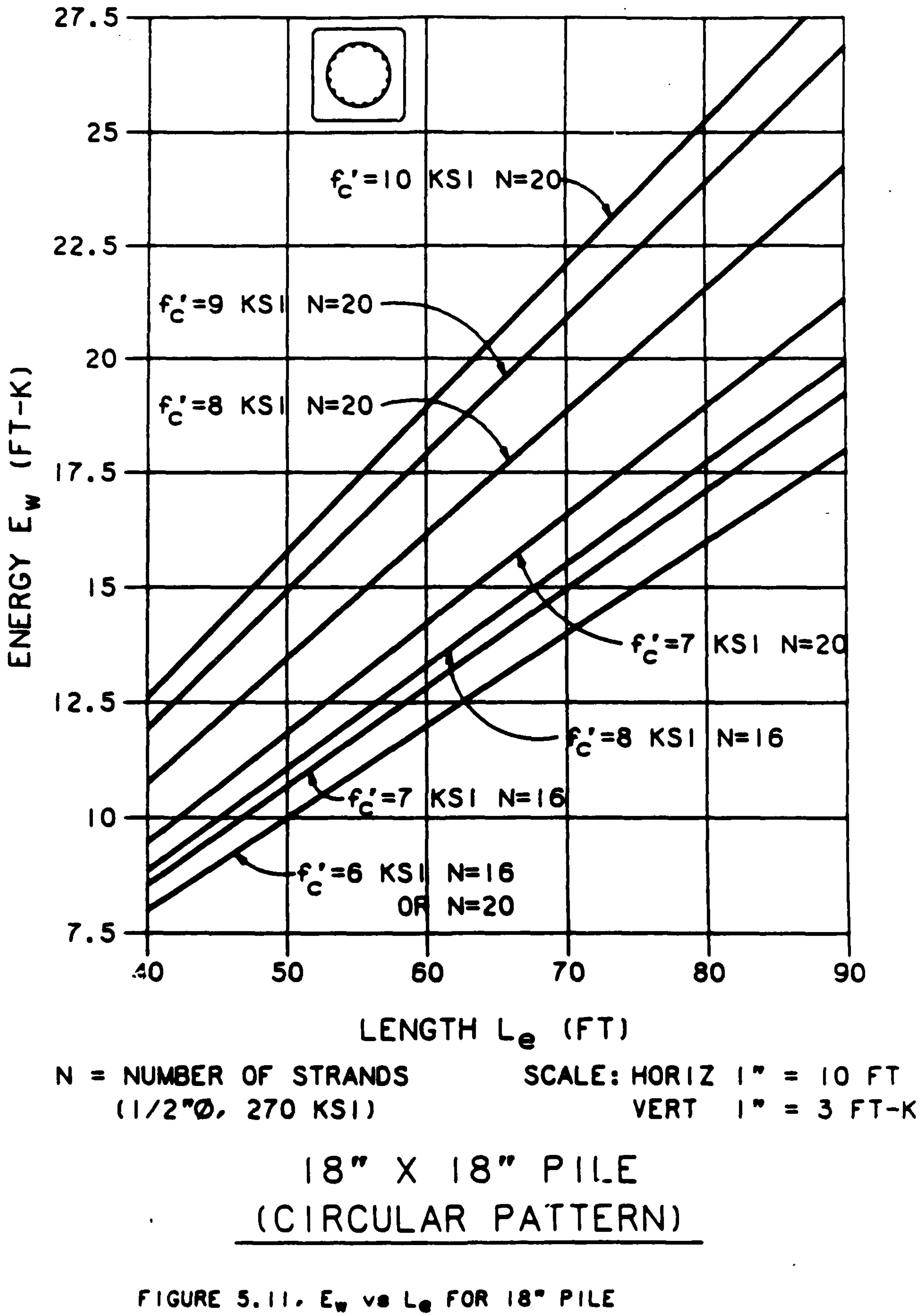
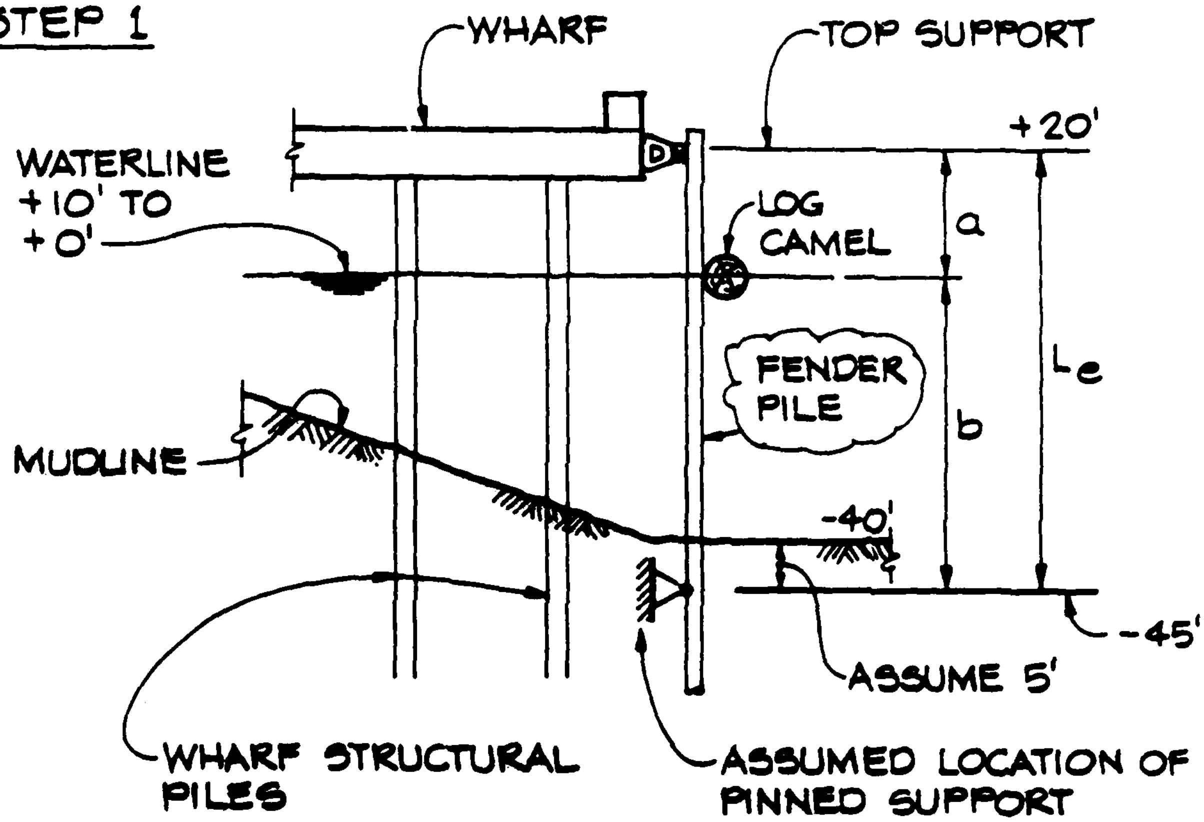


FIGURE 5.11. E_w vs L_e FOR 18" PILE W/ CIRCULAR STRAND PATTERN

DESIGN EXAMPLE

STEP 1



a) $L_e = 20' + 40' + 5' = \boxed{65'}$

b) a & b VARY, RANGES FROM :

$a = 20' - 10' = \boxed{10'}$ OR $a = 20' - 0' = \boxed{20'}$

$b = 65' - 10' = \boxed{55'}$ $\quad$ $b = 65' - 20' = \boxed{45'}$

c) FOR A PARTICULAR SHIP, USING THE NAVY DESIGN MANUAL, DM 25.1, THE TOTAL SHIP BERTHING ENERGY IS FOUND TO BE 120 FT-K.
BASED ON AN ELASTIC DISTRIBUTION OF THE TOTAL ENERGY INTO THE SYSTEM, THE MAXIMUM ENERGY REQUIRED BY ONE PILE : $\boxed{E_W = 19 \text{ FT-K}}$

STEP 2 IT HAS BEEN DECIDED THAT A PILE WITH A RECTANGULAR STRAND PATTERN WILL BE USED.

a) USING FIGURE 5.3:

PLOTTING $\left.\begin{array}{l} E_W = 19\ \text{FT-K} \\ L = 65' \end{array}\right\} \Rightarrow \boxed{\text{USE 18" □ PILE}}$

b) FOR 18" PILE, USE FIGURE 5.6

PLOTTING $\left.\begin{array}{l} E_W = 19\ \text{FT-K} \\ L = 65' \end{array}\right\} \Rightarrow \boxed{\text{USE } f'_c = 8\ \text{ksi},\ N = 20}$

STEP 3 FROM TABLE 5.2, FOR:

$\left.\begin{array}{l} 18\text{" □ PILE} \\ f'_c = 8\ \text{ksi} \\ N = 20 \end{array}\right\} \Rightarrow \begin{array}{ll} C_{ew} = .306 & M_{wc} = 325 \\ C_{en} = .491 & M_{nc} = 386 \end{array}$

a) CALCULATE PILE ENERGY CAPACITIES

$E_{wc} = .306 \times 65' = \boxed{19.9\ \text{FT-K}}$

$E_{nc} = .491 \times 65' = \boxed{31.9\ \text{FT-K}}$

b) CALCULATE RANGE OF LOAD ON PILE

$P_{wc}\ (\text{MAX}) = 325 \times 65'/10' \times 55' = \boxed{38.4\ \text{K}}$

$P_{wc}\ (\text{MIN}) = 325 \times 65'/20' \times 45' = \boxed{23.5\ \text{K}}$

$P_{nc}\ (\text{MAX}) = 386 \times 65'/10' \times 55' = \boxed{45.6\ \text{K}}$

$P_{nc}\ (\text{MIN}) = 386 \times 65'/20' \times 45' = \boxed{27.9\ \text{K}}$

STEP 4 CALCULATE THE MAXIMUM DEFLECTIONS.

$DEFL._{wc} = 1.5 \times 19.9/23.5 = 1.27' = \boxed{15.2''}$

$DEFL._{nc} = 1.5 \times 31.9/27.9 = 1.72' = \boxed{20.6''}$

STEP 5 CALCULATE REACTION @ TOP.

$R_{wc} = 38.4 \times (55/65) = \boxed{32.5K}$

$R_{nc} = 45.6 \times (55/65) = \boxed{38.6K}$

STEP 6 SIZE RUBBER FENDER @ TOP IF USED. THERE IS AN INTERACTION BETWEEN THE FENDER PILE & RUBBER FENDER. ITS CONTRIBUTION TO THE ENERGY CAPACITY OF THE FENDER SYSTEM SHOULD ALSO BE CONSIDERED.

USE $R_{wc} = \boxed{32.5K\ MAX}$

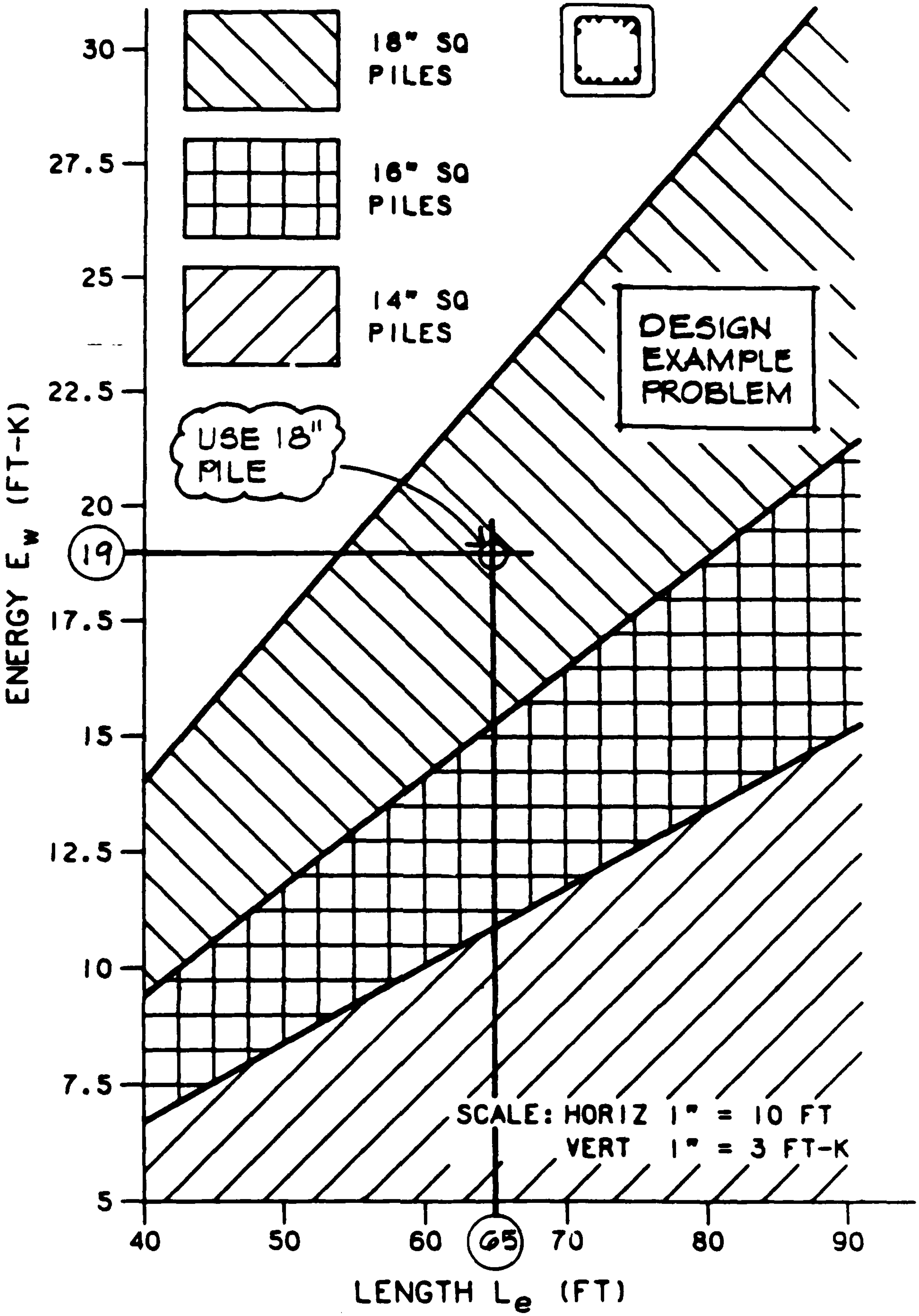


FIGURE 5.3. E vs L FOR 14", 16" & 18" PILES W/ RECTANGULAR STRAND PATTERNS

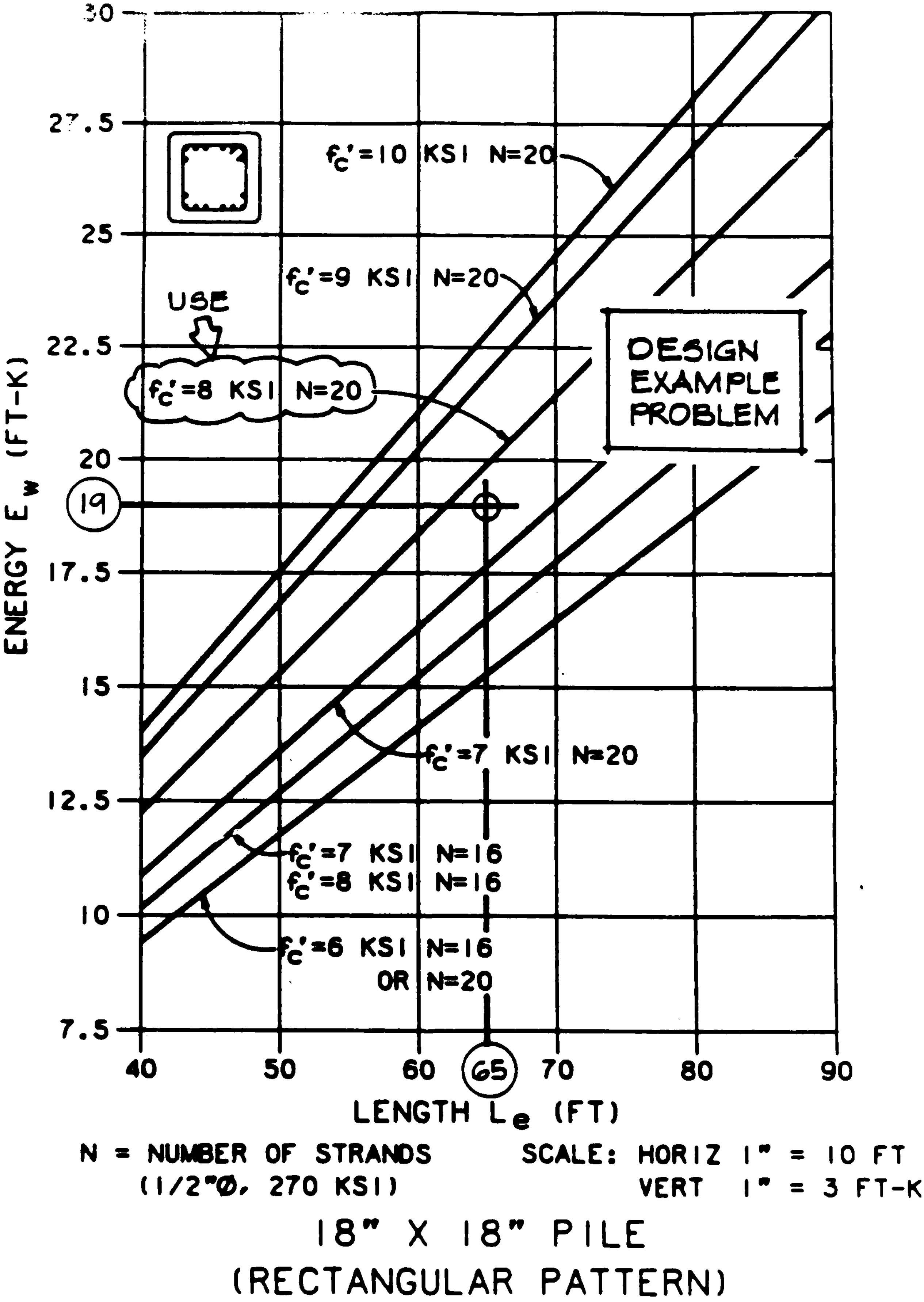


FIGURE 5.6, E vs L FOR 18" PILE
W/ RECTANGULAR STRAND PATTERN

TABLE 5.2
CONSTANTS FOR PRESTRESSED CONCRETE
FENDER PILES WITH RECTANGULAR STRAND PATTERN

Pile Configuration			C_{ew} $= E_w/L_e$	C_{en} $= E_n/L_e$	M_{wc}	M_{nc}
Size	f'_c	No. of Strands	(ft-k/ft)	(ft-k/ft)	(k-ft)	(k-ft)
14" sq	6	12	0.128	0.211	124	146
	6	16	0.128	0.211	130	153
	7	12	0.148	0.245	133	158
	7	16	0.148	0.245	139	167
	8	12	0.168	0.275	140	167
	8	16	0.168	0.275	148	179
16" sq	6	12	0.183	0.275	190	216
	6	16	0.183	0.305	200	239
	6	20	0.183	0.305	208	250
	7	12	0.194	0.305	196	226
	7	16	0.211	0.352	214	256
	7	20	0.211	0.352	224	271
	8	12	0.194	0.323	198	233
	8	16	0.235	0.389	226	270
	8	20	0.235	0.400	237	290
18" sq	6	16	0.235	0.362	273	315
	6	20	0.235	0.389	286	343
	7	16	0.254	0.400	284	330
	7	20	0.271	0.445	307	367
	8	16	0.254	0.428	289	342
	8	20	0.306	0.491	325	386
	9	20	0.337	0.529	340	402
	10	20	0.351	0.562	354	414

SECTION 6
UHMW RUBSTRIPS

The UHMW rubstrips with bolt holes down the center from the previous report were recommended for use in this phase [1.3]. Width and thickness requirements of the strip were investigated and refined. A steel pipe sleeved hole is recommended in the pile for rubstrip attachment because it would tend to carry the stress around the hole on the compression face and hence not affect the structural capacity of the pile.

To prevent abrasion damage to the concrete fender piles from vessel impact and vice versa, rubstrips can be installed on the outward face of the fender piles. These rubstrips will extend from the top of the pile and to approximately 3 ft below extreme low water.

Traditionally, rubstrips have been of dense wood construction, such as Greenheart or Balau timbers. In some installations, treated Douglas fir soft woods have been used. In each case, the effective life of the rub-strips has been limited because of physical damage from ship contact or deterioration from marine borer attack.

The use of UHMW material offers improvement to this problem. The material is inert and has high strength and stiffness. It has been used in facing panels for large fender systems in marine terminal facilities around the world. Figure 6.1 presents the recommended rubstrip details.

UHMW rubstrips would be attached to the fender piles in the traditional fashion of bolting through the center of the pile. Galvanized pipe sleeves are used in the pile to allow for easy removal and replacement of the strip should it be damaged.

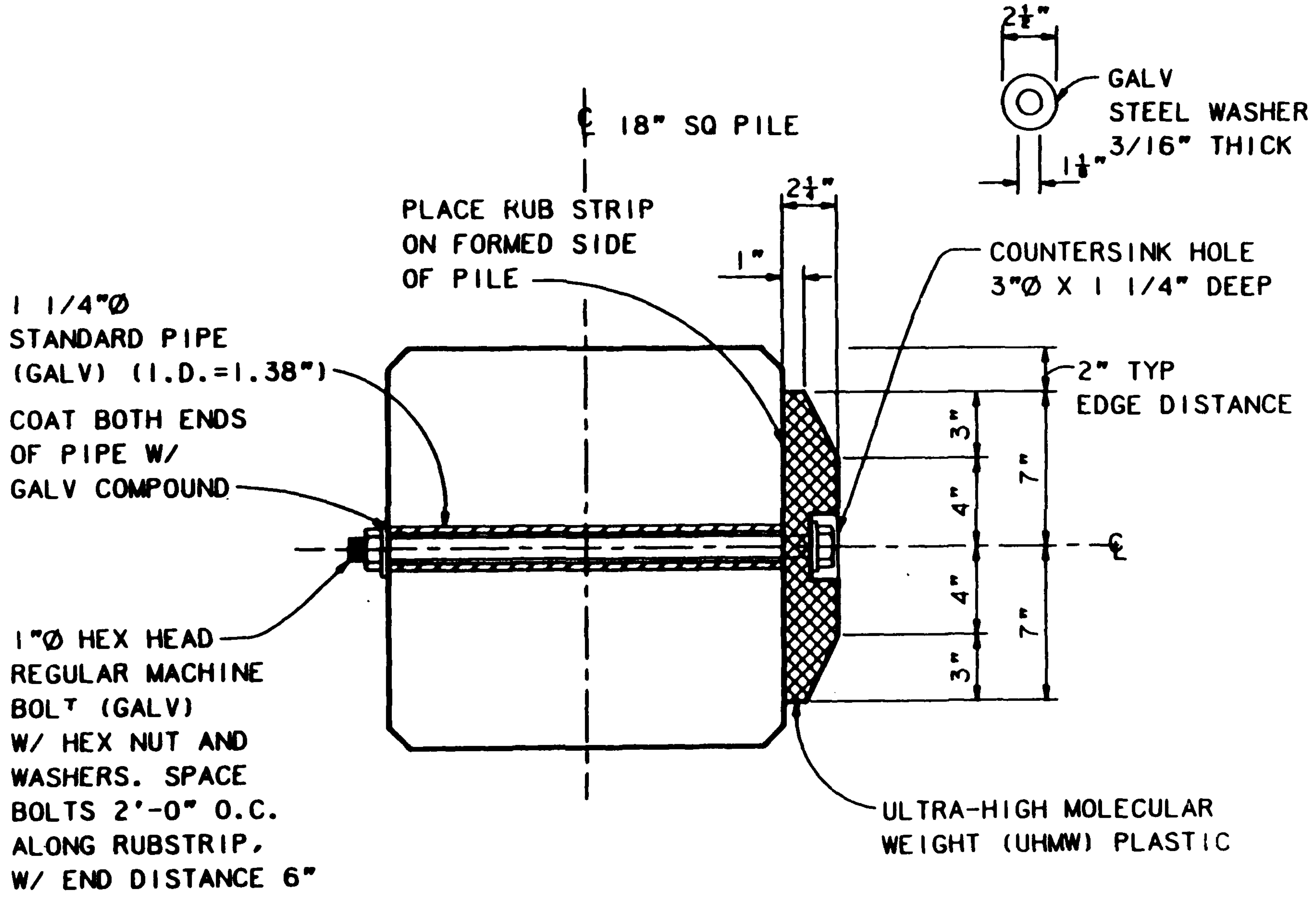


FIGURE 6.1, ATTACHMENT OF RUBBING STRIP

SECTION 7
COST ANALYSIS

7.1 BASELINE PILE COSTS

The cost analysis was based on the baseline piles from Section 4.3. Costs were determined for five key components: concrete, silica fume, prestressing strand, rebar, and formwork and curing. These costs were derived from local contractors' bids for concrete piling and discussions with precasters and concrete suppliers. Baseline pile costs are shown in Table 7.1.

Silica fume is a significant cost item of the pile, equal to the cost of the concrete itself. Silica fume is a very fine pozzolan which is typically added to the concrete at a rate of 10% by weight of cement to increase strength and durability. It is currently available throughout the United States in slurry form, which is expensive. There are also limited sources for dry, densified silica fume, which is only a fraction of the cost. Additional placement costs must also be assumed because the fineness of the silica fume tends to make the concrete "stickier" and harder to place.

High-strength and durable concrete is achievable in many areas of the country without the use of silica fume. Therefore, it is recommended that silica fume be deleted from the baseline pile except as required for strength and durability. This decision should be based on local expertise.

7.2 PILE COST COMPARISONS

Costs were determined for the different pile configurations in Section 5. The costs for the different pile configurations were based on the baseline pile (Table 7.1) and adjusted using unit prices for four key components -- concrete, prestressing strand, rebar, and formwork and curing. The following variables were evaluated:

- o Pile size
- o Concrete strength
- o Number of prestressing strands
- o Type of confinement steel (square or circular)

The previous variables are independent of pile length and were used to provide costs per foot of pile length. Fixed costs are discussed in Section 7.3.

a) Pile Size: Three pile sizes were investigated: 14, 16, and 18 in. sq. The concrete volumes were based on the nominal out-to-out dimensions and were not reduced by the volume of reinforcement contained. In addition, it was assumed that the pile size has no affect on the unit costs of the concrete, prestressing strand, or rebar.

b) Concrete Strength: Five concrete strengths were investigated: 6, 7, 8, 9, and 10 ksi. The major variable in concrete strength is cement content. The unit cost of the concrete, per cubic yard, for different concrete strengths was based on the assumption that $\pm$ one sack of cement equals a $\pm$1-ksi change in concrete strength from the baseline concrete. Concrete placement costs were calculated as a percentage of concrete material cost (= 36%). Larger pile size and less reinforcing steel can reduce the cost per volume of concrete but this was not taken into account.

c) Number of Prestressing Strands: Unit prices, per pound, for the prestressing strands were obtained from precasters. It was assumed that the configuration and amount of prestressing steel in a section would not affect the unit cost of the strands; 1/2-in.-diameter 270-ksi prestressing strands were assumed.

d) Type of Confinement Steel: Unit prices, per pound, for the reinforcement were obtained from steel fabricators and precasters. Two configurations were investigated: square and circular tied reinforcement. No. 3 rebar in a square tie or circular spiral was the least expensive material and W11 wire spiral was the most expensive material. Placement costs for either configuration were assumed equal; however, the circular configuration requires less material per foot of pile and would therefore cost less than the square configuration.

The results of the cost comparisons are shown in Table 7.2 for rectangular strand configurations and Table 7.3 for circular strand configurations. Two columns of cost data are provided, cost per lineal foot and cost per ft-kip of energy.

The most cost-effective pile (per ft-kip of energy) for a new installation is the largest size pile available with the highest strength concrete available and the minimum number of strands to provide a balanced condition (i.e., concrete strain $\leq$ 0.0021 in./in., strand stress (f_{pw}) $\leq$ 210 ksi). In locations where pile spacing is predetermined, the most cost-effective pile may be the one with the lowest cost per lineal foot, provided it has the required energy absorption capacity.

7.3 PILE SYSTEM COST COMPARISONS

The following fixed costs, per pile, were assumed to complete the estimate:

- o Driving cost equal to $700/pile (piles assumed to be between 40 and 90 ft in length and 14, 16, or 18 in. sq)
- o Rubstrips equal to $450/pile (length is assumed to be 14 ft, width equal to pile size minus 4 in.)

The baseline pile configurations were evaluated at a common span length of 65 ft (total length equal to 70 ft). The results are shown in Table 7.4.

TABLE 7.1
COST ESTIMATE

18-In.-Sq Baseline Pile	Material Cost	Labor Cost	Cost per Lin Ft
1a. Concrete (0.083 cu yd/lin ft)	$4.57	$1.68	$ 6.25
1b. Silica Fume (71 lb/cu yd)	5.31	1.08	6.39
2. Prestress Strand (20 1/2-in.-dia 270-ksi)	6.36	4.24	10.60
3a. Rebar (No. 3, sq pattern @ 3 in. on ctr = 8 lb/lin ft)	2.08	3.52	5.60
3b. Rebar(No. 3, circ pattern @ 3 in. on ctr = 5.5 lb/lin ft)	1.42	2.41	3.83
4. Formwork and Curing (4.5 sq ft/lin ft)	0.90	2.25	3.15
With silica fume, sq pattern (1a + 1b + 2 + 3a + 4)		$31.99 per lin ft	
With silica fume, circ pattern (1a + 1b + 2 + 3b +4)		$30.22 per lin ft	
Without silica fume, sq pattern (1a + 2 + 3a +4)		$25.60 per lin ft	
Without silica fume, circ pattern (1a + 2 + 3b +4)		$23.83 per lin ft	

Notes: 1) Costs based on West Coast pricing.

2) Costs are FOB precaster and do not include applicable taxes or general contractor's overhead and profit.

3) Costs for UHMW rubstrip embedments are not included.

TABLE 7.2
COMPARATIVE COSTS FOR RECTANGULAR STRAND PATTERN

Pile Configuration					
Size	f'_c	No. of Strands	Cost per Lineal Foot ($/L)	Cost per Ft-Kip Energy ($/C_{ew}$)	
14" sq	6	12	$16.47	$129.00	
	6	16	18.59	145.00	
	7	12	16.56	112.00	
	7	16	18.68	126.00	
	8	12	16.79	100.00	<
	8	16	18.91	113.00	
16" sq	6	12	18.54	101.00	
	6	16	20.66	113.00	
	6	20	22.78	124.00	
	7	12	18.70	96.00	
	7	16	20.82	99.00	
	7	20	22.94	109.00	
	8	12	18.99	98.00	
	8	16	21.11	90.00	<
	8	20	23.23	99.00	
18" sq	6	16	22.86	97.00	
	6	20	24.98	106.00	
	7	16	23.10	91.00	
	7	20	25.22	93.00	
	8	16	23.47	92.00	
	8	20	25.59	84.00	<
	9	20	25.97	77.00	
	10	20	26.35	75.00	<

Arrows indicate most cost-effective pile per size grouping.

TABLE 7.3
COMPARATIVE COSTS FOR CIRCULAR STRAND PATTERN

Pile Configuration					
Size	f'_c	No. of Strands	Cost per Linear Foot ($/L)	Cost per Ft-Kip Energy ($/C_{ew}$)	
14" sq	6	12	$15.01	$128.00	
	6	16	17.13	146.00	
	7	12	15.09	110.00	
	7	16	17.21	126.00	
	8	12	15.32	99.00	<
	8	16	17.44	113.00	
16" sq	6	12	16.92	109.00	
	6	16	19.04	123.00	
	6	20	21.16	131.00	
	7	12	17.08	105.00	
	7	16	19.20	104.00	
	7	20	21.32	115.00	
	8	12	17.38	102.00	
	8	16	19.50	93.00	<
	8	20	21.62	103.00	
18" sq	6	16	21.09	105.00	
	6	20	23.21	116.00	
	7	16	21.33	100.00	
	7	20	23.45	99.00	
	8	16	21.71	98.00	
	8	20	23.83	89.00	<
	9	20	24.20	81.00	
	10	20	24.58	78.00	<

Arrows indicate most cost-effective pile per size grouping.

TABLE 7.4
18-IN. SQ BASELINE PILE SYSTEM COSTS

Baseline Piles	Furnish[1]	Install	Total Cost[1]	$/Ft-Kips
f'_c = 6 ksi	$1749	$700	$2449	$160.00
f'_c = 7 ksi	1765	700	2465	140.00
f'_c = 8 ksi	1791	700	2491	125.00
f'_c = 9 ksi	1818	700	2518	115.00
f'_c = 10 ksi	1845	700	2545	112.00

<u>Notes</u>

1. Costs based on 70-ft pile length and square confinement pattern.
2. If rubstrips are required, add $450 per pile.
3. Costs do not include supports at the top of the pile.

SECTION 8
DRAWINGS AND SPECIFICATIONS

8.1 DESIGN DRAWINGS

Attached in Appendix A are four design drawings. A discussion of each drawing is presented below:

a) Drawing NCEL 87-14-1F: This design drawing is for the baseline pile with a square confinement pattern as described in Section 4.3. The designer must indicate the pile length (L).

b) Drawing NCEL 87-14-2F: This drawing presents a generic pile. The designer selects the pile size, reinforcing configuration, stressing data, and concrete strength. The designer must also indicate the pile length (L).

c) Drawing NCEL 87-14-3F: This drawing presents the possible reinforcing configurations with a rectangular strand pattern as used in the design aids. The available pile sizes are 14, 16, and 18 in. sq with the number of strands either 12, 16, or 20. Stressing data are provided for each pile size.

d) Drawing NCEL 87-14-4F: This drawing presents the possible reinforcing configurations with a circular strand pattern as used in the design aids. The available pile sizes are 14, 16, and 18 in. sq with the number of strands either 12, 16, or 20. Stressing data are provided for each pile size.

e) Drawing NCEL 87-14-5F: This drawing presents UHMW rubstrip and attachment details. The designer must select the width and length of the rubstrip and detail the pipe sleeves on Drawing NCEL 87-14-1F or NCEL 87-14-2F.

8.2 PRESTRESS LOSS

Prestress losses were calculated in accordance with Ref. 8.1. Initial jacking force given for each pile grouping is based on prestress losses calculated at 60 days. The effective concrete compressive stress (f_{pc}) at 60 days is assumed to be 600 psi.

The prestress losses in the pile can be broken down into three different effects: elastic shortening of the pile, shrinkage, and creep. The stress relaxation loss for strands jacked to a very low level of stress can be assumed to be zero. There are many different pile configurations, each one having a slightly different magnitude of loss.

Time-dependent prestress losses (shrinkage and creep) were calculated at 60 days for several reasons:

a) The maximum time between precasting and installation usually is 60 days. After installation, the piles will be in a wet or very moist (100% relative humidity) condition; hence, time-dependent losses will be greatly diminished or nonexistent after installation.

b) By 60 days, 50% to 60% of the time-dependent losses will have occurred. If losses at final are used, the piles may be too "stiff" and their energy absorbing capacity would be reduced. If losses are underestimated, the energy absorbing capacity of the piles would actually be increased.

8.3 SPECIFICATIONS

Specifications are included in Appendix B. These specifications are based on NAVFAC Guide Specification 02367, Prestressed Concrete Piling, with appropriate modifications to Parts 1 (General) and 2 (Products). Part 3 (Execution) is generally site-specific and should therefore be completed by the facility designer according to the General and Technical Notes attached to Guide Specification 02367.

REFERENCES

1.1 Naval Civil Engineering Command. Test and Evaluation Master Plan: Development of Prestressed Concrete Fender Piles, Program Element 63725N, Project No. Y0995-SL. Alexandria, VA, June 1984.

1.2 Naval Civil Engineering Laboratory. Technical Memorandum M51-87-08, Development of Prestressed Concrete Fender Piles - Final Pile Details: Tests and Recommendations. G. Warren and J. Malvar, Port Hueneme, CA, May 1987.

1.3 Naval Civil Engineering Laboratory. Contract Report: Prestressed Concrete Fender Piles - Analysis and Final Test Pile Detailing. ABAM Engineers Inc., Federal Way, WA, 21 March 1986.

2.1 Naval Civil Engineering Laboratory. Technical Memorandum 51-85-19, Development of Prestressed Concrete Fender Piles - Preliminary Tests.

3.1 Private communication with G. Warren, NCEL, supplementary test pile results, 4 August 1987.

3.2 American Concrete Institute (ACI). Building Code Requirements for Reinforced Concrete, ACI 318-83.

4.1 Naval Facilities Engineering Command (NAVFAC). DM 25.01, Piers and Wharves.

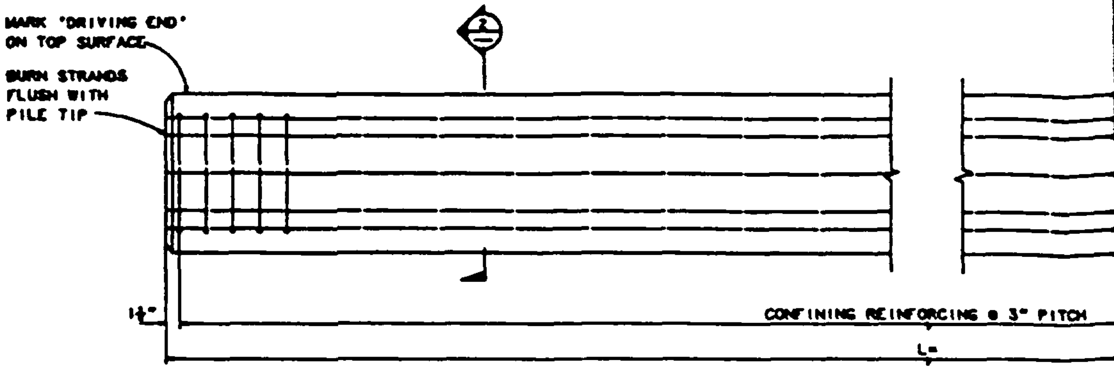
8.1 Prestressed Concrete Institute (PCI). PCI Design Handbook, 3rd edition.

NOTATIONS

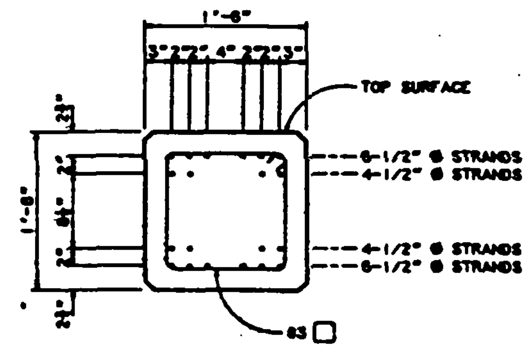
a	Distance from support at pile top to location of load
b	Distance from support at pile bottom to location of load
C_{en}	Coefficient used to calculate E_{nc} based on L
C_{ew}	Coefficient used to calculate E_{wc} based on L
E	Total berthing energy in fender system
E_f	Failure energy in the pile when pile has failed
E_{nc}	Nominal energy capacity in the pile when the maximum compressive strain in the concrete = 0.003 in./in.
E_s	Modulus of elasticity of prestressing strand
E_w	Maximum design working energy in one pile
E_{wc}	Working energy capacity in the pile based on the criteria in Section 3.1 (usually maximum concrete strain = 0.0021 in./in.)
f'_c	Specified compressive strength of concrete
f_{pc}	Concrete compressive stress due to effective prestress force, f_{se}
f_{se}	Effective stress in prestressing strand after accounting for losses
f_{pw}	Stress in prestressing strand at working energy
f_{ps}	Stress in prestressing strand at nominal energy
f_{pu}	Specified tensile strength of prestressing strand
f_{py}	Specified yield strength of prestressing strand
L	Length of pile
L_e	Pile design span length calculated from centerline of supports
M_{nc}	Moment in pile at location of load when energy = E_{nc}

M_{wc}	Moment in pile at location of load when energy = E_{wc}
P	Load on pile
P_{nc}	Load on pile at energy E_{nc}
P_{wc}	Load on pile at energy E_{wc}
R	Reaction of pile top on fender
R_{nc}	Reaction of pile top on fender due to P_{nc}
R_{wc}	Reaction of pile top on fender due to P_{wc}
$Defl_{nc}$	Deflection of pile at load location due to P_{nc}
$Defl_{wc}$	Deflection of pile at load location due to P_{wc}
ϵ_c	Concrete compression strain at extreme fiber

APPENDIX A
DRAWINGS



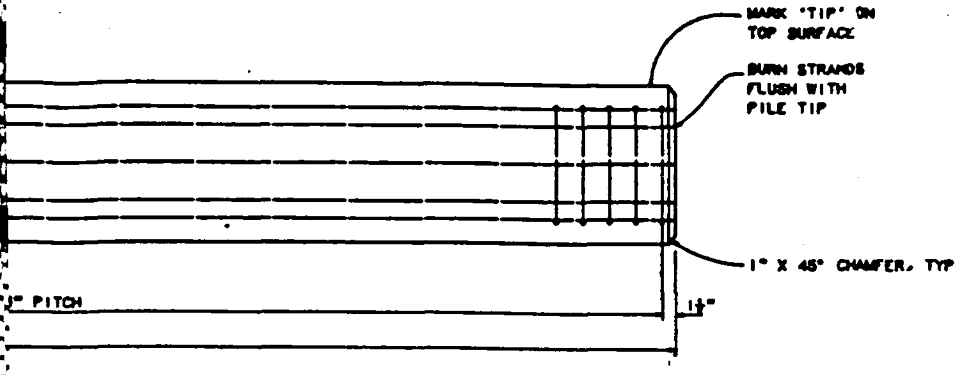
18" PRESTRESSED CONCRET
1 1/2"=1'-0"



NOTE: DIMENSIONS ARE SHOWN TO CENTERLINE OF STRAND. MAINTAIN 2" MIN CONCRETE COVER TO ALL STEEL.

PILE SECTION
1 1/2"=1'-0"

REVISIONS



NOTES:

1. LIFTING DEVICE LOCATIONS ARE TO BE DETERMINED BY THE FABRICATOR CONSIDERING HANDLING STRESSES AND DRIVING REQUIREMENTS.
2. PRECAST CONCRETE PILES
   f'c = 8000 PSI AT 28 DAYS
3. CONCRETE STRENGTH AT PRESTRESS TRANSFER, 3500 PSI MINIMUM.
4. REINFORCEMENT
   A. REBAR: ASTM A615, GR 60.
   B. PRESTRESSING STEEL: 1/2" Ø, 270 KSI SEVEN WIRE, UNCOATED, STRESS RELIEVED OR LOW RELAXATION STRANDS PER ASTM A416
5. MARK EACH END OF PILE AS SHOWN.

ONCRETE PILE

STRESSING DATA	
PILE SIZE	18" x 18"
CONCRETE AREA	324 IN²
TENSIONING UNITS	20 STRANDS
INITIAL PRESTRESS FORCE	227 KIPS
DESIGN PRESTRESS FORCE	194.4 KIPS
DESIGN CONCRETE STRESS	800 PSI

2.

IF SHEET IS LESS THAN 28"X40", SHEET HAS BEEN REDUCED. USE GRAPHIC SCALE.

1 1/2"=1'-0" 1' 9" 6" 3" 0 1' 2'

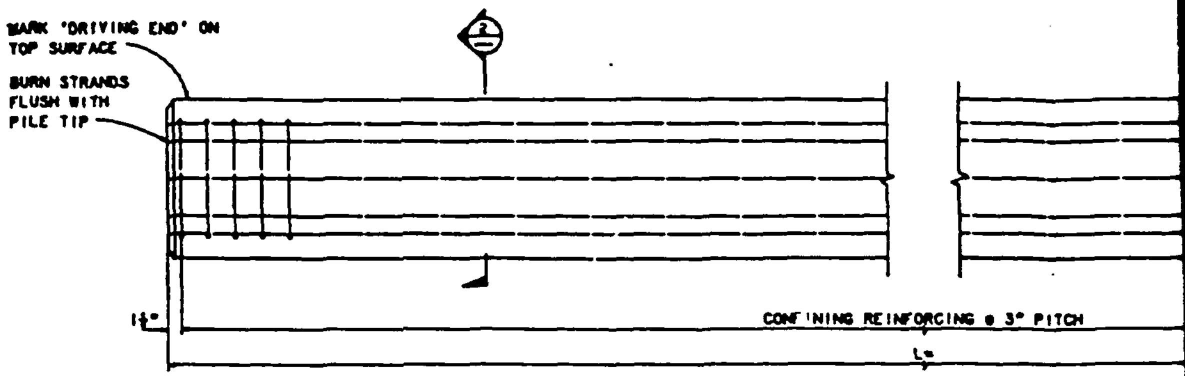
NCEL 87-14-1F

DEPARTMENT OF THE NAVY NAVAL FACILITIES ENGINEERING

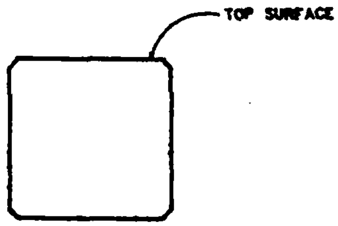
PRESTRESSED CONCRETE FENDER PIL

STANDARD PILE

F 80091

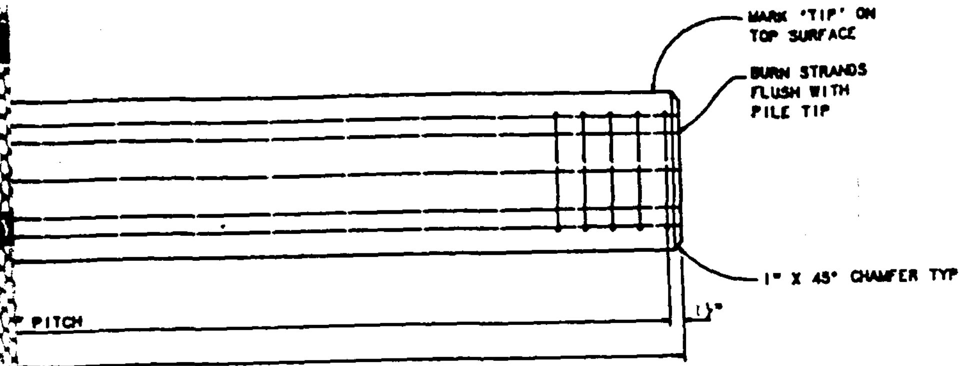


PRESTRESSED CONCRETE PILE
NTS



NOTE: DIMENSIONS ARE SHOWN TO CENTERLINE
OF STRAND. MAINTAIN 2" MIN
CONCRETE COVER TO ALL STEEL.

PILE SECTION
NTS



TE PILE

NOTES:

1. LIFTING DEVICE LOCATIONS ARE TO BE DETERMINED BY THE FABRICATOR CONSIDERING HANDLING STRESSES AND DRIVING REQUIREMENTS.
2. PRECAST CONCRETE PILES
   f'c =   PSI AT 28 DAYS
3. CONCRETE STRENGTH AT PRESTRESS TRANSFER, 3500 PSI MINIMUM.
4. REINFORCEMENT
   A. REBAR: ASTM A615, GR 60.
   B. PRESTRESSING STEEL: 1/2" Ø, 270 KSI SEVEN WIRE, UNCOATED, STRESS RELIEVED OR LOW RELAXATION STRANDS PER ASTM A416.
5. MARK EACH END OF PILE AS SHOWN.

STRESSING DATA	
PILE SIZE	
CONCRETE AREA	
TENSIONING UNITS	STRANDS
INITIAL PRESTRESS FORCE	
DESIGN PRESTRESS FORCE	
DESIGN CONCRETE STRESS	800 PSI

NCEL 87-14-2F

PRESTRESSED CONCRETE FENDER PIL

_____ INCH PILE

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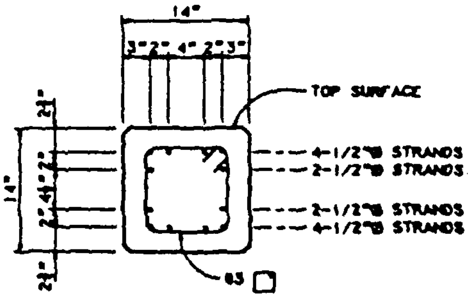
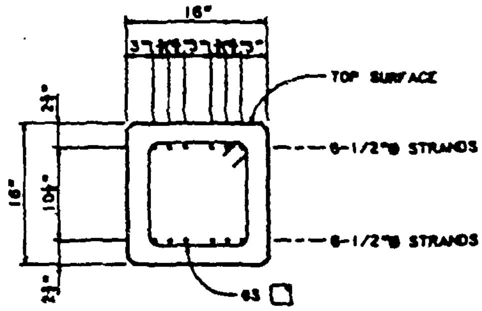
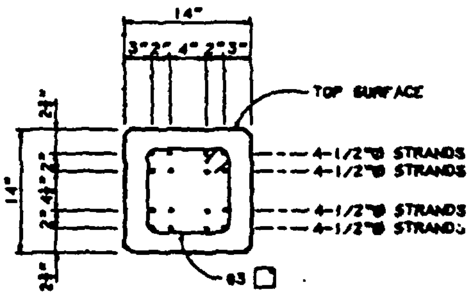


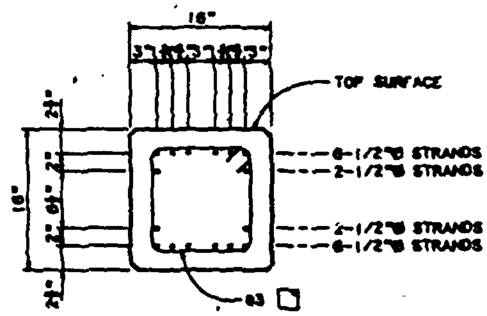
1 SECTION - 14" SQ PILE
1 1/2"=1'-0" W/ 12 STRANDS



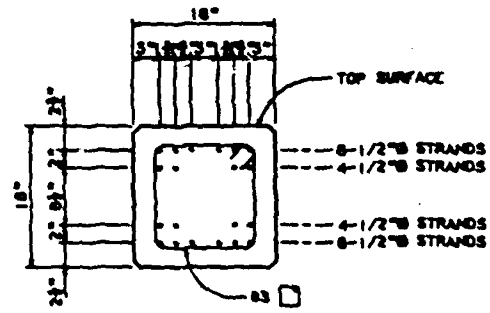
3 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 12 STRANDS



2 SECTION - 14" SQ PILE
1 1/2"=1'-0" W/ 16 STRANDS



4 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 16 STRANDS



5 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 20 STRANDS

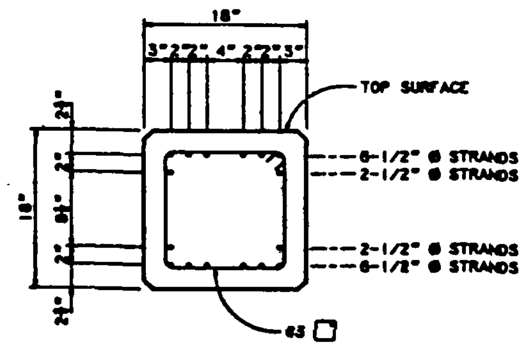
STRESSING DATA	
PILE SIZE	14" X 14"
CONCRETE AREA	196 IN^2
INITIAL PRESTRESS FORCE	144 KIPS
DESIGN PRESTRESS FORCE	117.6 KIPS
DESIGN CONCRETE STRESS	600 PSI

STRESSING DATA	
PILE SIZE	16" X 16"
CONCRETE AREA	256 IN^2
INITIAL PRESTRESS FORCE	183 KIPS
DESIGN PRESTRESS FORCE	153.6 KIPS
DESIGN CONCRETE STRESS	600 PSI

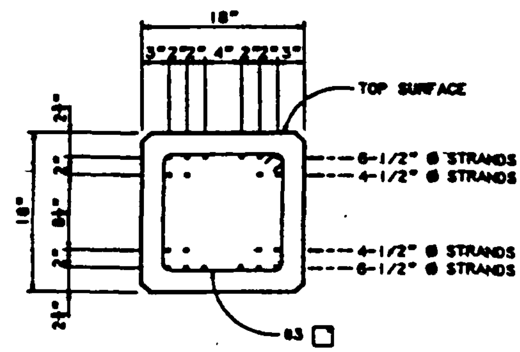
REVISIONS

NOTES:

1. DIMENSIONS ARE SHOWN TO CENTERLINE OF STRAND. MAINTAIN 2" MIN CONCRETE COVER TO ALL STEEL.



6 SECTION - 18" SQ PILE W/ 16 STRANDS
1 1/2"=1'-0"



7 SECTION - 18" SQ PILE W/ 20 STRANDS
1 1/2"=1'-0"

STRESSING DATA	
PILE SIZE	18" X 18"
CONCRETE AREA	324 IN²
INITIAL PRESTRESS FORCE	227 KIPS
DESIGN PRESTRESS FORCE	194.4 KIPS
DESIGN CONCRETE STRESS	800 PSI

1 1/2"=1'-0" 1' 9" 6" 3" 0 1' 2'

IF SHEET IS LESS THAN 28"X40", SHEET HAS BEEN REDUCED. USE GRAPHIC SCALE.

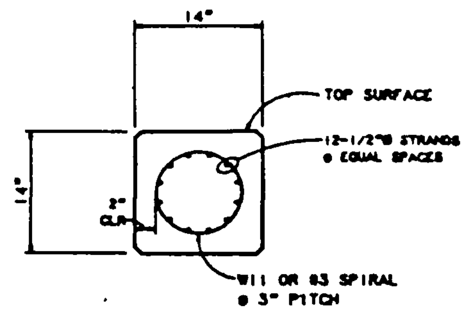
NCEL 87-14-3F

PRESTRESSED CONCRETE FENDER PILES

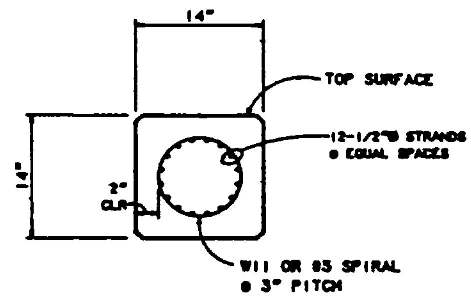
RECTANGULAR REINFORCEMENT CONFIGURATION

F 80091

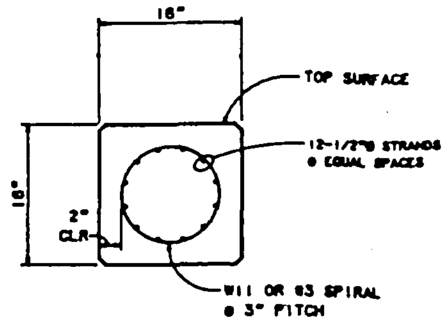
SCALE AS NOTED



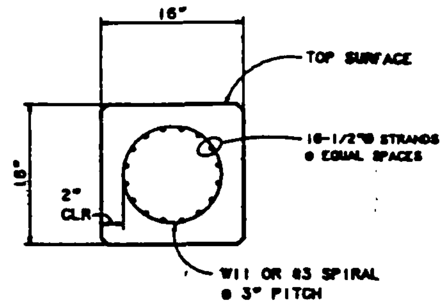
1 SECTION - 14" SQ PILE
1 1/2"=1'-0" W/ 12 STRANDS
CIRCULAR PATTERN



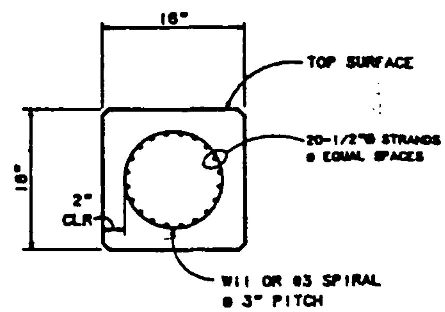
2 SECTION - 14" SQ PILE
1 1/2"=1'-0" W/ 16 STRANDS
CIRCULAR PATTERN



3 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 12 STRANDS
CIRCULAR PATTERN



4 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 16 STRANDS
CIRCULAR PATTERN



4 SECTION - 16" SQ PILE
1 1/2"=1'-0" W/ 20 STRANDS
CIRCULAR PATTERN

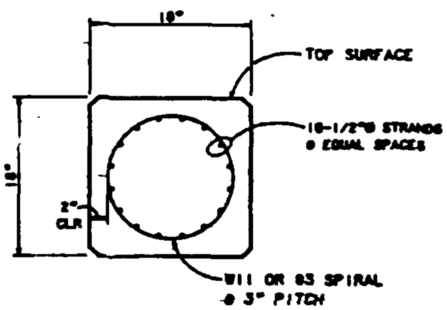
STRESSING DATA	
PILE SIZE	14" X 14"
CONCRETE AREA	196 IN^2
INITIAL PRESTRESS FORCE	144 KIPS
DESIGN PRESTRESS FORCE	117.6 KIPS
DESIGN CONCRETE STRESS	600 PSI

STRESSING DATA	
PILE SIZE	16" X 16"
CONCRETE AREA	256 IN^2
INITIAL PRESTRESS FORCE	183 KIPS
DESIGN PRESTRESS FORCE	153.6 KIPS
DESIGN CONCRETE STRESS	600 PSI

REVISIONS

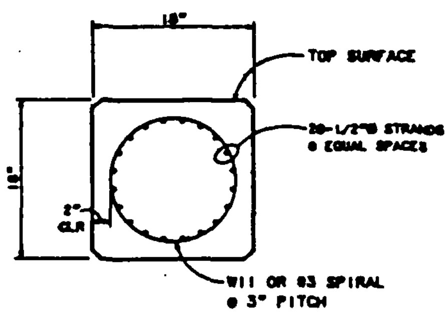
NOTES:

1. DIMENSIONS ARE SHOWN TO CENTERLINE OF STRAND. MAINTAIN 2" MIN CONCRETE COVER TO ALL STEEL.
2. REINFORCEMENT
   A. REBAR: ASTM A615, GR 60
   B. SPIRAL REINFORCEMENT: W11 COLD DRAWN WIRE REINFORCEMENT AS PER ASTM A82, F_y = 70 KSI



SECTION - 18" SQ PILE W/ 16 STRANDS CIRCULAR PATTERN

1 1/2"=1'-0"



SECTION - 18" SQ PILE W/ 20 STRANDS CIRCULAR PATTERN

1 1/2"=1'-0"

STRESSING DATA	
PILE SIZE	18" X 18"
CONCRETE AREA	324 IN²
INITIAL PRESTRESS FORCE	227 KIPS
DESIGN PRESTRESS FORCE	194.4 KIPS
DESIGN CONCRETE STRESS	600 PSI

1 1/2"=1'-0" 1' 9" 6" 3" 0 1' 2'

IF SHEET IS LESS THAN 28"X40", SHEET HAS BEEN REDUCED. USE GRAPHIC SCALE.

NCEL 87-14-4F

PRESTRESSED CONCRETE FENDER PILES

CIRCULAR REINFORCEMENT CONFIGURATION

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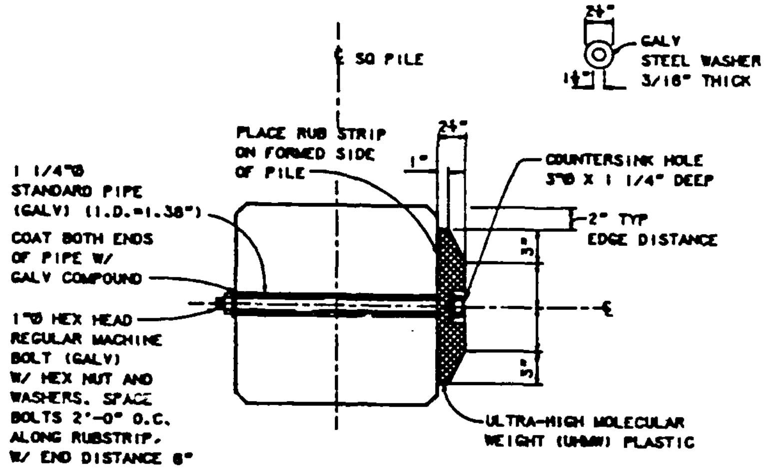

DETAIL - RUB STRIP
NTS

REVISIONS

2

NCEL 87-14-5F

PRESTRESSED CONCRETE FENDER PILES

RUB STRIP DETAILS

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AS NOTED

APPENDIX B
SPECIFICATIONS

PRESTRESSED CONCRETE FENDER PILING
GENERAL NOTES

1. This specification is based on NFGS-02367, Prestressed Concrete Piling, with appropriate modifications for use with prestressed concrete fender piling.

2. Paragraph 1.2.5 -- Insert the number of copies of quality control procedures

3. Paragraph 2.2.1 -- Insert the specified compressive strength, f'_c.

4. Part 3, Execution, should be reviewed by a geotechnical engineer familiar with the immediate project area to determine if jetting or driving should be used to install the piles to the indicated butt elevation.

5. The following information should be shown on the drawings:

   a. Locations and design energy of piles

   b. Size, shape, and length of piles

   c. Pile butt elevation and minimum embedment depth

   d. Pile-to-fender-system attachment details

   e. Rub strip length, pipe sleeve spacing, and attachment details, if used

   f. Internal jet pipe length and details, if used.

   g. Concrete compressive strength

   h. Locations, sizes, and number of prestressing strands and stressing data

   i. Soil data

SPECIFICATIONS FOR
PRESTRESSED CONCRETE FENDER PILING

PART 1 - GENERAL

1.1 **APPLICABLE PUBLICATIONS** -- The publications listed below form a part of this specification to the extent referenced. The publications are referred to in the text by the basic designation only.

1.1.1 American Association of State Highway and Transportation Officials and Prestressed Concrete Institute (AASHTO-PCI) Publications:

STD 112-61	Standard Prestressed Concrete Piles

1.1.2 American Concrete Institute (ACI) Publications

ACI 211.1-81 (R85)	Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete
ACI 212.1R-81 (R86)	Admixtures for Concrete
ACI 214-77 (R83)	Recommended Practice for Evaluation of Strength Test Results of Concrete
ACI 315-80	Details and Detailing of Concrete Reinforcement
ACI 318-83	Building Code Requirements for Reinforced Concrete

1.1.3 American Society for Testing and Materials (ASTM) Publications

A 53-84	Pipe, Steel, Black, and Hot-Dipped, Zinc-Coated Welded and Seamless
A 82-85	Cold-Drawn Steel Wire for Concrete Reinforcement
A 153-82	Zinc Coating (Hot-Dip) on Iron and Steel Hardware
A 416-85	Uncoated Seven-Wire, Stress-Relieved Strand for Prestressed Concrete
A 421-80 (R85)	Uncoated Stress-Relieved Wire for Prestressed Concrete
A 501-84	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing

A 615-85	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 616-85	Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
A 617-84	Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
C 31-84	Making and Curing Concrete Test Specimens in the Field
C 33-84	Concrete Aggregates
C 39-84	Compressive Strength of Cylindrical Concrete Specimens
C 143-78	Slump of Portland Cement Concrete
C 150-85	Portland Cement
C 172-71 (R77)	Fresh Concrete Sampling
C 260-77	Air-Entraining Admixtures for Concrete
C 311	Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete
C 494-82	Chemical Admixtures for Concrete
C 618-85	Fly Ash and Raw or Calcined Natural Pozzolan for Use as A Mineral Admixture in Portland Cement Concrete

1.1.4 Prestressed Concrete Institute (PCI) Publications

MNL-116-85	Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products

1.2 SUBMITTALS

1.2.1 Quality Control -- As an exception to the quality control requirements specified elsewhere herein, all approvals specified herein under this section will be performed by the Contracting Officer.

1.2.2 Shop Drawings -- Submit for piles showing placement of the reinforcing steel, prestressing steel, lifting inserts, and all other embedded items. Show size and type of strands, strand force at release, and final effective strand force. Show the use of special embedded or attached lifting devices, the employment of pickup points, support points other than pickup points, or any other method of pickup on the shop drawings. Drawings shall conform to ACI 315.

1.2.3 Contractor-furnished Mix Design -- At least 30 days prior to concrete placement, submit a concrete mix design for each type of concrete used for the piles.

1.2.4 Certified Laboratory Test Reports -- Before delivery of materials, submit certified copies of the reports of all tests required in referenced publications or otherwise specified herein. The testing shall have been performed within one year of submittal of test reports for approval by an approved independent laboratory. Test reports on a previously tested material shall be accompanied by notarized certificates from the manufacturer certifying that the previously tested material is of the same type, quality, manufacture, and make as that proposed for use in this project. Certified test reports, mill certificates, or certificates of compliance are required for the following:

a. Aggregates -- Certified test reports for the following tests specified in ASTM C 33:

   (1) Grading
   (2) Amount of material finer than No. 200 sieve
   (3) Organic impurities
   (4) Soundness
   (5) Clay lumps and friable particles
   (6) Coal and lignite
   (7) Weight of slag
   (8) Abrasion of coarse aggregate
   (9) Fineness modulus
   (10) Reactive aggregates
   (11) Freezing and thawing

b. Admixtures

c. Prestressing Steel

d. Reinforcing Steel

e. Portland Cement -- The certification shall identify the cement by brand name, type, mill location, quantity to be used, size of lot represented by quality control sample, lot number, and destination of shipment.

f. Fly Ash -- Provide fly ash test results performed within 6 months of submittal date.

g. Silica Fume -- Provide silica fume test results performed within 6 months of submittal date.

h. Concrete Mix Designs -- Certify, using a government-approved independent commercial testing laboratory, that the mix has been proportioned in accordance with ACI 211.1 or ACI 318 for the specified strength and is based on aggregate data which has been determined by laboratory tests during the last 12 months.

1.2.5 Quality Control Procedures -- Submit _____ copies of quality control procedures established, in accordance with PCI Manual MNL-116, by the precasting manufacturer.

1.2.6 Pile Driving Equipment -- Submit data on type and model of crane; type and length of leads; type of jetting equipment; type of drilling or spudding equipment; and type and size of hammer, helmets, pile cushion, and hammer cushion. Also submit information on pile installation sequence and pile installation procedures.

1.3 REQUIREMENTS

1.3.1 Piling Lengths and Quantity - Provide prestressed pretensioned concrete piles. Base bids upon the number, size, and length of piles as indicated. Adjustments in the contract price will not be made for cutting off piles or for broken, damaged, or rejected piles.

PART 2 - PRODUCTS

2.1 MATERIALS

2.1.1 Cement -- ASTM C 150, Type II, low alkali. Use cement with a tricalcium aluminate (C_3A) content of 6 to 10%. Alkalies as Na_2O shall be less than 0.60.

2.1.2 Water -- Water shall be potable and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be harmful to concrete or steel. Water shall not contain chloride ion.

2.1.3 Aggregates -- ASTM C 33, except as modified herein, free from any substance which may be deleteriously reactive with the alkalies in the cement in an amount sufficient to cause excessive expansion of the concrete. Do not mix, store in the same stockpile, or use fine aggregates from different sources of supply alternately in the same concrete mix or the same structure without approval. Maximum coarse aggregate size shall be 3/4 in.

2.1.4 Admixtures -- Do not use admixtures containing chlorides

a. Water-reducing Admixtures -- Use Type A or D of ASTM C 494. If required, use Type F or G of ASTM C , high-range water-reducing admixtures.

b. Air-entraining Admixtures -- Use ASTM C 260.

2.1.5 Mineral Admixtures

a. Fly Ash -- Use Class F fly ash of ASTM C 618. Fly ash content shall not exceed 25 percent by weight of total cementitious material.

b. Silica Fume -- If required, use silica fume from any approved source. Basis for approval shall be mill tests and manufacturer's certificate of compliance with ASTM C 618 and the following:

(1)	Specific gravity	2.0 - 2.4
(2)	Percent passing 325 mesh sieve	>99.0
(3)	SiO_2 content, percent	>90.0
(4)	Carbon content, percent	<5.0
(5)	Fe_2O_3 content, percent	<1.5
(6)	Total alkalies, percent	<1.0
(7)	Loss on ignition (LOI), percent	<2.0

2.1.6 Prestressing Steel -- Use seven-wire stress-relieved or low-relaxation strand conforming to ASTM A 416 with a guaranteed minimum ultimate tensile strength of 270 ksi. Use prestressing steel free of grease, oil, wax, paint, soil, dirt, and loose rust. Do not use prestressing strands or wire having kinks, bends, or other defects.

2.1.7 Reinforcing Steel -- ASTM A 615, A 616, or A 617, Grade 60, deformed reinforcing bars.

2.1.8 Ties and Spirals -- Steel for smooth wire spirals, ASTM A 82. Steel for individual ties, deformed reinforcing steel, ASTM A 615.

2.1.9 Pipe Sleeves -- Use ASTM A 53, Grade B, or ASTM A 501 galvanized pipe. Sleeves to be galvanized as per ASTM A 153 with chromate wash. Do not place galvanized pipe in contact with any prestressing or reinforcing steel.

2.2 CONCRETE

2.2.1 Contractor-Furnished Mix Design -- The concrete shall have a specified compressive strength, f'_c, of _____ psi. The minimum cement content shall be 600 lbs per cu yd of concrete. The design shall be prepared in accordance with ACI 211.1 or ACI 318, Chapter 4. ACI 318, Chapter 4, Table 4.3.2.2, shall be modified for a specified compressive strength, f'_c, over 5000 psi to permit a required average compressive strength, f'_{cr}, of f'_c + 700 psi. The concrete may be proportioned in accordance with ACI 214 for the probability of one test in ten falling below the specified compressive strength, f'_c, provided that the mix design reflects actual concrete plant standard deviations and the resulting production concrete conforms to the specified requirements. The mix-design shall be based on current materials previously evaluated by the concrete producer whose established methods of statistical quality control is in conformance with ACI 318. In the absence of such data, the

Contractor shall sample and test the aggregates for the design of concrete.

2.2.2 Mix Design Proportioning

a. Water/cement ratio shall be equal to or less than 0.40. If fly ash is used, the water/cement ratio shall be calculated as the weight of water divided by the weight of cement plus 60 percent of the weight of fly ash. If silica fume is used, the water/cement ratio shall be calculated as the weight of water divided by the weight of cement plus the weight of silica fume.

b. Mineral admixtures shall be used. Use a minimum fly ash content of 15 percent by weight of cement or a minimum silica fume content of 5 percent by weight of cement.

c. Maximum aggregate size shall not exceed 3/4 in.

d. Air-entrainment shall be 5 to 8 percent. Determine air void structure in accordance with ACI 212.1R. Spacing factor shall be less than 0.01 in., the specific surface area shall be greater than 600 sq in. per cubic inch of air void volume, and the number of air voids per inch of traverse shall be significantly greater than the numerical value of the percentage of air in the concrete.

2.2.3 Trial mixtures having proportions and consistencies of the proposed mix design shall be made to document the contractor's ability to produce workable concrete which does not segregate or show excessive slump loss characteristics.

2.3 FABRICATION OF PRETENSIONED PILES

2.3.1 General -- Piles shall be pretensioned concrete piles. Workmanship shall conform to standard commercial practice in prestressing plants.

2.3.2 Formwork -- Provide forms of metal, well braced and stiffened against deformation, accurately constructed, watertight, and supported on unyielding casting beds. Forms shall permit movement of the pile without damage during release of the prestressing force. Dimensional tolerances for manufacturing of piles shall be as follows:

a.	Length	3/8 in. per 10 ft of length.
b.	Cross Section	+1/2 in. to -1/4 in.
c.	Deviation from straight lines	Not more than 1/8 in. per 10 ft. of length.

d.	Pile head	±1/4 in. per foot of head dimension from true right angle plane. Surface irregularities -- ±1/8 in.
e.	Location of reinforcing steel	
	(1) Main reinforcement cover	1/8 to 1/4 in.
	(2) Spacing of spiral	±1/2 in.
f.	Location of pipe sleeves from true position	±3/8 in.

2.3.3 Pretensioning -- Anchorages for tensioning the prestressing steel shall be a type approved by the Contractor. Measure the tension to which the steel is to be pretensioned by the elongation of the steel and verify by the jack pressure reading on a calibrated gage. The gage shall have been calibrated within the last 6 months by a laboratory approved by the Contracting Officer. Provide means for measuring the elongation of the steel to at least the nearest 1/8 in. When the difference between the results of measurement and gage reading is more than 3 percent, determine the cause of the discrepancy and correct. Give the tensioning steel a uniform prestress prior to being brought to design prestress. Induce the same initial prestress in each unit when several units of prestressing steel in a pile are stretched simultaneously.

2.3.4 Casting

a. Convey concrete from the mixer to the forms as rapidly as practicable by methods which will not cause segregation or loss of ingredients. Deposit concrete as nearly as practicable to its final position in the forms. At any point in conveying, the free vertical drop of the concrete shall not exceed 3 ft. Clean conveying equipment thoroughly before each run. Remove concrete which has segregated in conveying.

b. Perform concrete casting within 3 days after pretensioning the steel; however, do not deposit concrete in the forms until the placement of reinforcement and anchorages has been inspected and approved by the pile manufacturer's quality control representative. Produce each pile of dense concrete straight with smooth surfaces with the reinforcement retained in its proper position during fabrication. Vibrator head shall be smaller than the minimum distance between steel for pretensioning. The plane of the heads of piles shall be perpendicular to the axis of the pile. Chamfer the ends of all piles and the corners of square piles 1 in. Do not remove concrete piles from the forms until the concrete has attained a compressive strength of at least 3500 psi.

2.3.5 Curing of Piles -- Moist or heat cure piles.

a. Moist Curing -- Moist cure using water, moist burlap coverings, plastic sheeting, or membrane curing compound for a minimum period of 7 days and until the concrete has reached a compressive strength equal to or greater than 70 percent of the specified compressive strength, f'_c.

b. Radiant Heat and Moisture Curing and Steam Curing -- After placement of concrete, moist cure for a period of 4 hours. Heat cure for a minimum period of 12 hours. Enclose the casting bed with a suitable enclosure. During application of heat, increase the air temperature at a rate not to exceed 60 degrees F per hour. Cure, at a maximum temperature of 160 degrees F until the concrete has reached the specified release strength. Reduce the temperature at a rate not to exceed 60 degrees F per hour until a temperature of 20 degrees F above ambient air temperature is reached. Venting of steam hoods may be necessary to reduce internal temperature to 20 degrees F above ambient.

2.3.6 Detensioning -- Perform releasing of prestressed steel in pretensioned piles in such an order that eccentricity of prestress will be a minimum. Release tension in the strands from the anchorage gradually. Do not release the stress after casting without approval by the pile manufacturer's quality control representative. Perform the transfer of prestressing force when the concrete has reached a compressive strength of not less than 3500 psi. Following the detensioning and cut-off of the strands, coat or seal the ends of the strands with rust-prohibiting compound.

PART 3 - EXECUTION

3.1 PILE DRIVING

3.1.1 Fixed lead pile drivers shall be used when installing all piles. Hanging or swinging leads shall not be used unless they are so constructed that they can be held in fixed position during driving operation. Piles shall not be driven until 100 percent of design strength has been attained and until at least 14 days after detensioning. Drive piles to the indicated butt elevation and to the minimum embedment depth shown on the drawings. During the initial driving and until the pile tip has penetrated beyond layers of very soft soil or below the bottom of predrilled or prejetted holes, use a reduced rated driving energy of the hammer of not more than 20,000 foot-pounds per blow or as otherwise directed by the Contracting Officer. If a pile fails to reach the indicated butt elevation or minimum embedment, the Contractor shall notify the Contracting Officer and perform corrective measures as directed.

3.1.2 Installation of Piles -- Take care to avoid damage to the piles during handling, placing the pile in the leads, and during the pile-driving operations. Inspect piles when delivered, when in the leads immediately before driving, and after installation. No visible cracks will be permitted. Notify the Contracting Officer of any visible cracks and perform corrective measures as directed. Laterally support piles during driving, but do not unduly restrain from rotation in the leads. Take special care to maintain the pile orientation during driving. Square the top of the pile to the longitudinal axis of the pile.

3.1.3 Pile Hammers -- Furnish a hammer having a capacity at least equal to the hammer manufacturer's recommendation for the total weight of pile and character of subsurface material to be encountered. Obtain the required driving energy of the hammer, except for diesel hammers, by use of a heavy ram and a short stroke with low-impact velocity. The driving energy of the hammer, at final driving, shall be not less than 30,000 foot-pounds. At final driving, operate the pile hammer in accordance with the manufacturer's recommendation for driving either end bearing piles or friction piles. At final driving, operate diesel-powered hammers at the rate recommended by the manufacturer for hard driving. Maintain sufficient pressure at the steam hammer so that (1) for double-acting hammer, the number of blows per minute during and at the completion of driving of a pile is equal approximately to that at which the hammer is rated; (2) for single-acting hammer, there is a full upward stroke of the ram; and (3) for differential-type hammer, there is a slight rise of the hammer base during each downward stroke.

3.1.4 Driving Helmets and Cushion Blocks

a. Driving Helmets and Pile Cushions -- Use a steel driving helmet or cap, including a pile cushion between the top of the pile and the driving helmet or cap, to prevent impact damage to the pile. The driving helmet or cap-and -pile - cushion combination shall be capable of protecting the head of the pile, minimize energy absorption and dissipation, and transmit hammer energy uniformly over the top of the pile. The driving helmet or cap shall fit sufficiently loose around the top of the pile so that the pile may be free to rotate without binding within the driving helmet. The Contractor shall demonstrate to the satisfaction of the Contracting Officer that the equipment to be used on the project performs the above function. The pile cushion shall be of laminated construction using softwood boards with the grain parallel to the end of the pile. The thickness of the pile cushion shall be 10 in. minimum. Contractor shall submit to the Contracting Officer at least 2 weeks before the start of pile driving operations detailed drawings of the driving helmet and pile cushion to be used.

b. Hammer Cushion or Capblock -- Use a hammer cushion or capblock between the driving cap and the hammer ram consisting of aluminum and micarta (or equal) discs stacked alternately in a steel housing. Use steel plates at the top and bottom of the capblock. Replace aluminum or micarta discs that have become damaged, split, or deteriorated in any manner. Do not replace capblock during the final driving of any pile. Under no circumstances will the use of small wood blocks, wood chips, rope, or other material permitting excessive loss of hammer energy be permitted. The Contractor shall submit to the Contracting Officer at least 2 weeks before the start of pile driving operations, detailed drawings of the hammer cushion or capblock including records of the successful use.

3.1.5 Tolerances in Driving -- Drive all piles with a variation of not more than 1 percent from the vertical for plumb piles. Check axial alignment of pile and leads at start of pile driving and when the pile top is approximately 5 ft above the indicated elevation. Make intermediate checks of pile alignment if there is evidence of pile drifting. If subsurface conditions cause pile drifting beyond the allowable axial alignment tolerance, notify the Contracting Officer and perform corrective measures as directed. Place butts within 2 in. of the location indicated. Manipulation of piles within specified tolerances is permitted, but do not manipulate piles more than 1-1/2 percent of their exposed length above the mudline. Check all piles for heave. Redrive, to the indicated elevation, piles found to be heaved.

3.1.6 Jetting and Predrilling -- If predrilling is used, the diameter of the drilled hole shall not exceed the diagonal dimension of the pile and the hole shall be kept open until the pile is inserted and advanced to the bottom of the hole. Jetting of the pile to obtain penetration is permitted. Discontinue jetting or predrilling at a depth of 5 ft from the indicated butt elevation, and achieve the remaining penetration by driving. Before starting the driving of the final 5 ft, firmly seat the piles in place by the application of a number of reduced-energy hammer blows. The Contractor shall arrange to provide an ample supply of water at adequate pressure for effective jetting.

3.1.7 Splices -- Splicing of piles is not permitted.

3.1.8 Buildups -- Buildups are not permitted.

3.1.9 Pile Cutoffs -- Cut off piles with a smooth level cut using pneumatic tools, sawing, or other suitable methods approved by the Contracting Officer. The use of explosives for cutting is not permitted.

3.2 FIELD INSPECTION AND TESTS

3.2.1 Product Quality Control -- Where piling is manufactured in a plant with an established quality control program as attested to by a current certification in the PCI "Certification Program for Quality Control," perform product quality control in accordance with PCI Manual MNL-116. Where piling is manufactured by specialists or in plants not currently enrolled in the PCI "Certification Programs for Quality Control," set up a product quality control system in accordance with PCI Manual MNL-116 and perform concrete and aggregate quality control testing using an independent commercial testing laboratory approved by the Contracting Officer in accordance with the following. Submit test results to the Contracting Officer.

a. Aggregate Tests -- Take samples of the fine and coarse aggregate at the concrete batch plant and test. Perform mechanical analysis (one test for each aggregate size) including determination of the specific gravity. Tabulate the results of the tests in accordance with ASTM C 33.

b. Strength Tests -- Sample concrete in accordance with ASTM C 172 at the time the concrete is deposited for each production line. Compression tests shall conform to methods of ASTM C 39 and C 31. Perform slump tests in accordance with ASTM C 143. Mold six cylinders each day or for every 20 cu yd of concrete placed, whichever is greater. Perform strength tests 28 days after molding the cylinders, except test two cylinders of the set at 7 days or 14 days, or at a time for establishing the transfer of prestressing force (release strength) and removal of pile from the forms. Cure the cylinders in the same manner as the piles and place at the point where the poorest curing conditions are offered. This is at the coolest point in the bed for steam curing. Cylinders to be tested at 28 days shall be moist cured.

c. Changes in Proportions -- If, after evaluation of strength test results, the compressive strength falls below the specified compressive strength, make adjustments in the proportions and water content and changes in the temperature, moisture, and curing procedures as necessary to secure the specified strength. Submit all changes to the Contracting Officer in writing.

d. Strength Test Results -- Evaluate compression test results at 28 days in accordance with ACI 214 using a coefficient of variation of 20 percent. Evaluate the strength of concrete by averaging the test results (two specimens) of each set (four specimens) of standard cylinders tested at 28 days. Not more than 10 percent of the individual specimens tested shall have an average compressive strength less than the specified ultimate compressive strength.

3.2.2 Pile Records -- For each pile, keep a record of the number of blows required for each foot of penetration and the number of blows for the last 6-in. penetration or fraction thereof. Include in the record the type and size of the hammer used, the rate of operation, the stroke or equivalent stroke for the diesel hammer, the type of driving helmet, and the type and dimension of the hammer cushion (capblock) and the pile cushion used. Record any unusual occurrence during driving of the pile. Include in the record performance characteristics of jet pump, unassisted penetration of pile, jet-assisted penetration of pile, and tip elevation before driving and at end of driving. The Contractor shall notify the Contracting Officer 10 days prior to driving of piles. Submit complete and accurate records of installed piles to Contracting Officer within 15 calendar days after completion of the pile driving. Make pile-driving records available to the Contracting Officer at the job site a minimum of 24 hours after each day of pile driving.

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